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(1) B.S.

LEVEL #

Name Of Dam: BEDFORD RESERVOIR DAM

Location: BEDFORD COUNTY, VIRGINIA

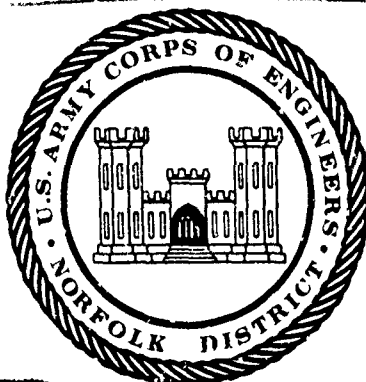
Inventory Number: VA 01904

(6) **PHASE I INSPECTION REPORT**  
**NATIONAL DAM SAFETY PROGRAM**

Bedford Reservoir Dam, Inventory Number:  
VA-01904. Bedford County, Virginia.  
Phase I Inspection Report.

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(10) James A. Walsh



(9) Final rept.

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## 20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

## PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of the Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the design flood should not be interpreted as necessarily posing a highly inadequate condition. The design flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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PHASE I REPORT

NATIONAL DAM SAFETY PROGRAM

Name of Dam: Bedford Reservoir Dam, #VA 01904  
State: Virginia  
County: Bedford County  
USGS Quad Sheet: Peaks of Otter  
Stream: Stony Creek  
Date of Inspection: 1 December 1978

Bedford Reservoir Dam is a zoned earthfill structure about 550 feet long and 54 feet high. The dam is owned and operated by the city of Bedford, Virginia, and supplies water to the city of Bedford. The dam is classified as an intermediate size with a significant hazard classification. The principal spillway consists of a 6-foot wide and 5-foot high concrete tunnel running through the dam at low level. The tunnel is served by three 16-inch intake levels and a 30-inch by 30-inch drawdown sluice gate located in the upstream portion of the intake structure. Water passes into the tunnel through a 30-inch by 30-inch discharge sluice gate which remains partially open. Water also flows through a 16-inch water supply pipe by means of gravity to a filtration plant several miles downstream. The dam is located on Stony Creek about 3.4 miles upstream of Kelso Mill, Virginia.

Based on criteria established by the Department of the Army, Office of the Chief of Engineers (OCE), the spillway is rated as adequate. The spillway will pass the Spillway Design Flood, which is considered the 1/2 PMF for this dam, without overtopping the dam.

The visual inspection revealed no apparent problems and there are no immediate needs for remedial measures. It is recommended within 12 months that stability analyses be performed on the upstream slope assuming a minimum drawdown from normal pool. It is also recommended that a regular maintenance program be initiated to maintain the integrity of the structure and correct those minor deficiencies listed in Section 7.

Submitted By: Original signed by  
JAMES A. WALSH

JAMES A. WALSH, P.E.  
Chief, Design Branch

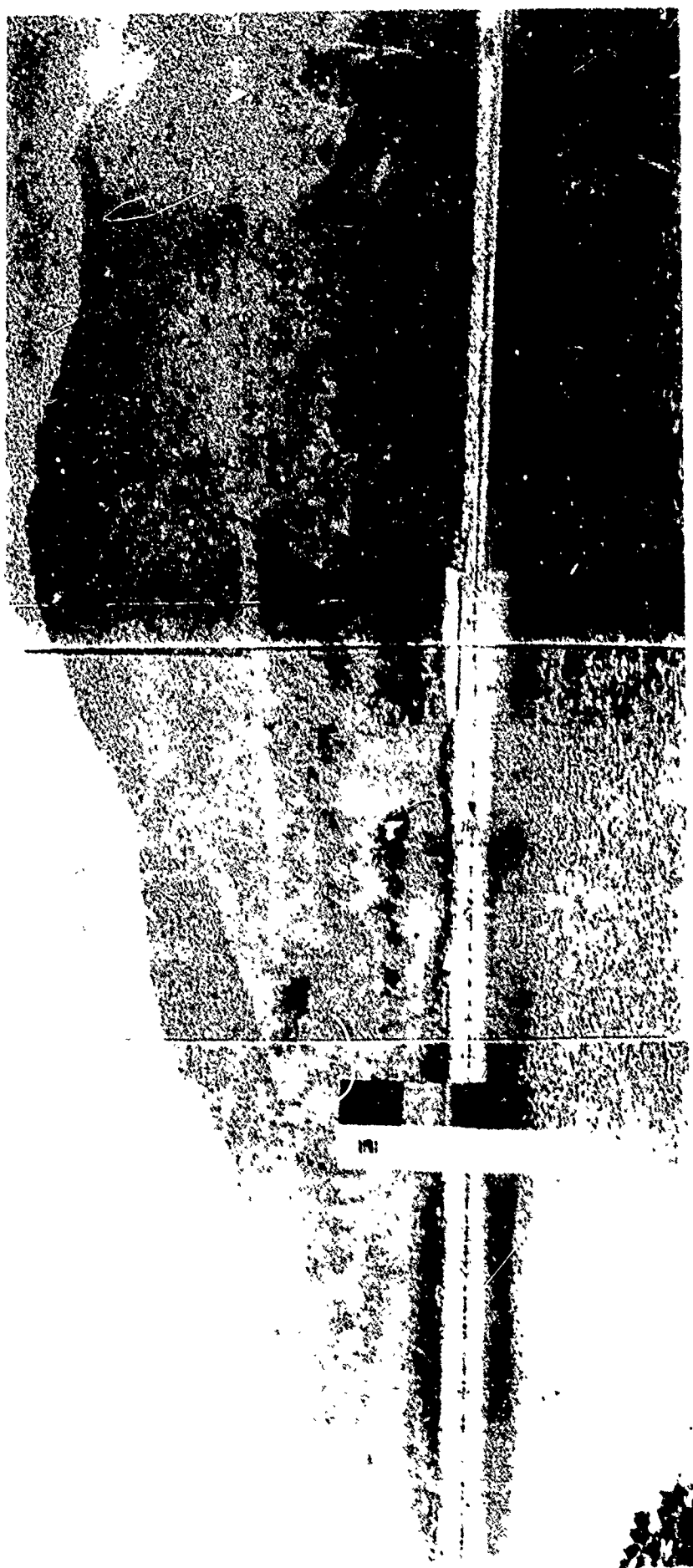
Approved By: Original signed by:

Douglas L. Haller  
DOUGLAS L. HALLER  
Colonel, Corps of Engineers  
District Engineer

Recommended By:  
Original signed by  
ZANE M. GOODWIN

ZANE M. GOODWIN, P.E.  
Chief, Engineering Division

Date: FEB 26 1979



OVERVIEW OF DAM FROM RESERVOIR



## SECTION 1

### PROJECT INFORMATION

#### 1.1 General:

1.1.1 Authority: Public Law 92-367, 8 Aug 72, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams (Appendix VII, Reference 4). The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

#### 1.2 Project Description:

1.2.1 Dam and Appurtenances: Bedford Reservoir Dam is an earth embankment dam about 550 feet long and 54 feet high. The top of the dam is 20 feet wide with an elevation of 1378.0 feet m.s.l. The upstream slope has 1 vertical to 3 horizontal (1:3) slope with riprap protection from elevation 1358.0 to 1371.0 feet m.s.l. The downstream slope has a (1:2.5) slope and is benched at elevations 1363.5, 1348.5, and 1333.5 to accommodate a surface drainage system consisting of catch basins and underground conduits.

The embankment has an impervious core keyed into foundation bedrock and an intricate foundation drainage system consisting of blanket and toe drains. Plan views and profiles of the embankment, with the surface and foundation drains, are shown on Drawings 2 and 14, Appendix I, respectively.

The principal spillway consists of a 6-foot wide and 5-foot high concrete tunnel running through the dam at low level. The tunnel is served by three 16-inch intake levels (elevations 1360, 1347, 1334). The top two intakes are always open allowing water into the intake structure. A 30-inch by 30-inch discharge sluice gate is partially opened discharging into the tunnel from the intake structure to allow discharge into the stream below the dam. Water flows from the intake structure by gravity through a 16-inch water supply pipe located within the tunnel. This water flows to a filtration plant several miles downstream.

The emergency spillway is a concrete side-channel spillway located at the right of the embankment. It has a bottom width of about 140 feet with a crest elevation 1369.0 (including 1 foot of wood flashboards) and side slopes of 1:1. The approach channel is constructed of riprap that covers the 40 feet adjoining the spillway crest. The 495-foot long discharge channel is constructed of concrete with several 2.5 inch hydrostatic relief plugs and one 14-inch drain that controls flow from areas to the right of the spillway.

A 30-inch by 30-inch sluice gate with the invert at elevation 1325.75 is located on the upstream side of the intake structure. This intake allows water to flow into the intake structure, then permits withdrawal of water from the bottom of the reservoir if the 30-inch by 30-inch discharge sluice gate is open.

1.2.2 Location: Bedford Reservoir Dam is located on Stony Creek about 3.4 miles upstream of Kelso Mill, Virginia. The reservoir formed by the dam is known locally as Stony Creek Reservoir.

1.2.3 Size Classification: The dam is classified as an "intermediate" size structure because of height (54 feet).

1.2.4 Hazard Classification: The dam is located in a rural area and is therefore given a significant hazard classification in accordance with guidelines contained in Section 2.1.2 of Reference 4, Appendix VII. The hazard classification used to categorize dams is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 Ownership: City of Bedford, Virginia.

1.2.6 Purpose: Water supply for the city of Bedford, Virginia.

1.2.7 Design and Construction History: The dam was designed by Wiley and Wilson, Inc. and constructed in 1958 by English Construction Company. Flashboards were placed in the ungated spillway in 1966 and concrete was placed adjacent to the flashboards on the discharge channel side in 1975.

1.2.8 Normal Operational Procedures: Water is withdrawn from the reservoir through a three level intake system to supply the city of Bedford, Virginia. A 30-inch by 30-inch discharge sluice gate is partially open to allow flow downstream. Otherwise, regulation of flows is largely automatic with water rising above the crest of the emergency spillway passing freely downstream.

### 1.3 Pertinent Data:

1.3.1 Drainage Area: The dam controls a drainage area of 6.02 square miles.

1.3.2 Discharge at Dam Site:

Maximum Known Flood - approximately 1,000 c.f.s.

Emergency Spillway

Pool Level at Top of Dam - 12,150 c.f.s.

1.3.3 Dam and Reservoir Data: Pertinent data on the dam and reservoir are shown in the follow table:

Table 1.1 DAM AND RESERVOIR DATA

Item	Elevation Elevation feet, m.s.l.	Area, acres	Reservoir Capacity		Length, miles
			Acre- feet	Watershed, inches	
Top of dam	1378.0	40	760	2.4	.4
Emergency spillway crest	1369.0	28	455	1.4	.3
Streambed	1315+	-	-	-	-

## SECTION 2

### ENGINEERING DATA

2.1 Design: Design data was provided by the owners and by the designers, Wiley and Wilson, Consulting Engineers, Lynchburg, Virginia. The data reviewed included the following:

a. As-built drawings indicating plans, elevations, and sections of the dam and appurtenant structures. Also shown on the drawings are borrow pit and test boring locations (May 1, 1954).

b. "Preliminary Memorandum on Geological Conditions at a Proposed Dam Site on Stony Creek, New Bedford, Virginia," by Byron N. Cooper, February 23, 1953, Appendix IV.

c. "Report on Earth Resistivity Surveys at Proposed Dam Site on Stony Creek, Near Bedford, Virginia," by Byron N. Cooper, Appendix V. This report covers the investigation and recommendations for embankment materials.

d. Test boring logs in addition to those shown on the drawings, by Farwell Well Corporation, April 8, 1954, Appendix V.

e. Hydrologic data, and hydraulic design calculations, Appendix VI for the design of the emergency spillway.

f. Stability Analyses, Appendix VI. Copies of existing data are on file with the Norfolk District for future references.

2.2 Construction: There are no available construction records.

2.3 Evaluations: The as-built drawings and design calculations were adequate for review. No construction reports were available to evaluate construction methods or alterations. However, as-built drawings were available during the inspection and external structures appear to be in accordance to the drawings.

## SECTION 3

### VISUAL INSPECTION

#### 3.1 Findings:

3.1.1 General: The results of the 1 December 1978 inspection are recorded in Appendix III. At the time of the inspection the pool elevation was 1362.75, 6.25 feet below normal pool elevation. The outlet work was releasing a very small flow by way of a partially opened discharge sluice gate in the intake structure. Prior to the field inspection performed on 1 December 1978, the only known inspection was an underwater inspection of the intake structure performed by Virginia State Police divers. In general, no obstructions or siltation problems were found.

3.1.2 Dam: The embankment is in good condition, but is thickly vegetated with brush and 2-inch locus trees. No sloughing, erosion, or misalignment was noted. Several of the catch basins on the dam were blocked with debris.

3.1.3 Appurtenant Structures: Observations of the intake structure were made from the embankment and no deterioration was noted. The discharge tunnel appeared to be in good condition, but a 2-inch layer of iron bacteria was noted on the floor of the tunnel.

3.1.4 Emergency Spillway: The spillway is in good condition except for the following deficiencies: portions of the flashboards are missing near the right abutment, a tree is growing out of one joint, the riprapped discharge channel has eroded away on the left side, the right side is overgrown with vegetation, and debris has blocked the right wall subsurface drain.

3.1.5 Reservoir Area: The reservoir area did not have any shoreline erosion or apparent slope failures.

3.1.6 Downstream Channel: No erosion was observed in the downstream area. No homes are in the flood plain. A 25-foot high concrete dam is located approximately 2,000 feet downstream. Because of siltation this dam has very little storage capacity.

3.2 Evaluation: Overall the dam appears to be in good condition. The embankment should be trimmed of the brush and all trees cut at the trunk. The catch basins should be cleaned out. The tree in the emergency spillway should be removed. The erosion of the channel downstream of the spillway should be riprapped. Also, the vegetation and debris located in the right side should be removed, and the right drain should be uncovered and allowed to flow freely. An annual maintenance and inspection program with records should be initiated.

## SECTION 4

### OPERATIONAL PROCEDURES

4.1 Procedures: The maximum storage pool is elevation 1369.0, which is the top of the flashboards, on the emergency spillway crest. The reservoir provides the water supply for the city of Bedford, Virginia, approximately 7 miles downstream. Under normal flow conditions, the reservoir pool level is at the ungated spillway crest (elevation 1369.0). Three water intake elevations 1360, 1347, and 1334, of which the higher two are always open, allow water to flow into the intake structure. Water is withdrawn from the intake structure by means of one 16-inch water supply pipe and the partially opened discharge sluice gate. A 30-inch by 30-inch discharge sluice gate connects the intake structure to the discharge tunnel through the dam. The sluice gate is left partially opened to allow discharge into the stream below the dam.

4.2 Maintenance: A routine maintenance program has not been established for the Bedford Reservoir Dam, although periodic maintenance has occurred. Twice a year copper sulfate is added to the reservoir, while until recent years, every 2 or 3 years ground maintenance was performed by the Public Works Department to control tree and vegetation growth on the embankment and in the spillway. A representative of the city of Bedford visually checks the reservoir elevation and discharge characteristics at least once a week.

4.3 Warning System: At the present time, there is no warning system or evacuation plan in operation.

4.4 Evaluation: The dam does not require an elaborate operational and maintenance procedure. However, an annual maintenance and inspection program should be initiated to help detect and control problems that may occur.

## SECTION 5

### HYDRAULIC/HYDROLOGIC DESIGN

5.1 Design: Design calculations for the dam and spillway are on file at the owner's office.

5.2 Hydrologic Records: None were available.

5.3 Flood Experience: The maximum observed flood reached was approximately 1 foot high in the ungated spillway or elevation 1370.0.

5.4 Flood Potential: The Probable Maximum Flood (PMF), 1/2 PMF, and 100-Year Flood were developed and routed through the reservoir by use of the HEC-1DB computer program (Reference 1, Appendix VII) and appropriate unit hydrograph, precipitation, and storage-outflow data. Clark's Tc and R coefficients for the local drainage area were estimated from basin characteristics. The rainfall applied to the developed unit hydrograph was obtained from U.S. Weather Bureau Publications (Reference 2 and 3, Appendix VII). Losses were estimated at an initial loss of 0.80 inch and a constant loss thereafter of 0.05 inch/hour.

5.5 Reservoir Regulation: Pertinent dam and reservoir data are shown in Table 1.1.

Regulation of flow from the reservoir has been set to allow water to enter the upper two 16-inch intakes and collect within the intake structure. Flow occurs through a 16-inch water supply pipe to the filtration plant and occurs through the partially open discharge sluice gate. Water also flows past the dam over the emergency spillway in the event water in the reservoir rises above elevation 1369.0.

The storage curve supplied by the owners was extended above the top of the flashboards by use of U.S. Geological Survey Quadrangle Maps. Rating curves were developed for the emergency spillway, non-overflow section of dam, and the drawdown outlet. In routing hydrographs through the reservoir, it was assumed that the initial pool level was at the emergency spillway crest (top of flashboards). Flow through water supply pipe was neglected during routing.

5.6 Overtopping Potential: The probable rise of the reservoir and other pertinent information on reservoir performance is shown in the following table:

Table 5.1 RESERVOIR PERFORMANCE

Item	Normal Flow	Hydrograph		
		100-Year <sup>1/</sup>	1/2 PMF	PMF <sup>2/</sup>
Peak flow, c.f.s.				
Inflow	6	6,767	13,483	26,966
Outflow	6	5,894	12,937	26,700
Maximum elevation, ft., m.s.l.	-	1374.4	1377.4	1381.1
Emergency spillway (el. 1369.0)				
Depth of flow, ft.	-	5.4	8.4	12.1
Duration, hours	-	16.5	73.0	79.0
Velocity, f.p.s.	-	7.0	8.7	10.4
Non-overflow section (el. 1378.0)				
Depth of flow, ft.	-	-	-	3.1
Duration, hours	-	-	-	2.5
Velocity, f.p.s.	-	-	-	5.4
Tailwater elevation, ft., m.s.l.	1316+	-	-	-

<sup>1/</sup> The 100-Year Flood (1 percent Exceedence Frequency Flood) has 1 chance in 100 of being exceeded in any given year.

<sup>2/</sup> The Probable Maximum Flood is an estimate of flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

5.7 Reservoir Emptying Potential: A 30-inch by 30-inch gated outlet and a 16-inch outlet at elevation 1325.25 is available for dewatering the reservoir. The low level opening through the intake structure will permit withdrawal of about 305 c.f.s. with the reservoir level at the crest of the emergency spillway and essentially dewater the reservoir in less than 1 day.

5.8 Evaluation: Based on the size (intermediate) and hazard classification (significant) the recommended Spillway Design Flood is 1/2 PMF. Based on the risk involved in this project, it is considered that 1/2 PMF is appropriate as a Spillway Design Flood.

Conclusions pertain to present day conditions. The effect of future development on the hydrology has not been considered.



## SECTION 6

### DAM STABILITY

6.1 Foundation and Abutments: The preliminary memorandum by Bryon N. Cooper, provided in Appendix IV, covers the area geology, including a summary of the test borings shown on the drawings. Generally, the area geology consists of shallow residual soils overlying a massive dark-colored greenish-gray bedrock. The bedrock is composed largely of hornblende and plagioclase gneiss. However, there is a thick river deposit through the dam site consisting of an imbricated mass of river cobbles attaining a maximum thickness of about 27 feet. The preliminary report includes specific recommendations on keying impervious earth core into bedrock beneath the cobble layer. The drawings provide a profile of the dam foundation and ungated spillway on Drawings 9 and 7, Appendix I, respectively.

#### 6.2 Embankment:

6.2.1 Materials: The embankment consists of three zones of materials. The impervious core is classified "Class A" with "Class B and C" as the upstream and downstream ballast, respectively. The earth resistivity survey by Cooper provided in Appendix V covers the investigation and recommendations for embankment materials. The test borings by Farwell Well Corporation are believed to be some of the additional borings requested by Cooper in his report. Available data does not specify the characteristics of the classes of materials. Based on Cooper's survey report, the Class A is probably a residual plastic clay and the Class B is the above soil, too granular and rocky for use as in Class A clay. There is no reference in any of the information as to the nature of the Class C soil.

6.2.2 Stability: The embankment stability was performed according to Engineering For Dams, Volume III, by Justin, Hinds, and Gregor. Design parameters of  $\phi = 24^\circ$  and  $C = 0$  were assumed for the core material. The analyses were performed for the following loading conditions:

<u>Condition</u>	<u>Factor of Safety</u>
1. Resisting Force Against Headwater Pressure	3.8
2. Horizontal Shear in Downstream Portion 1V:2.5H Slope	1.49
3. Horizontal Shear in Upstream Portion (Sudden Drawdown, Worst Condition) 1V:3H Slope 1V:3.5H Slope	1.0 1.18

Based on the analyses, an upstream slope of 1V:3.5H and downstream slope of 1V:2.5H were recommended for design. A copy of the calculations are provided in Appendix VI.

6.2.3 Seismic Stability: The dam is located in Seismic Zone 2. Therefore, according to the Recommended Guidelines for Safety Inspection of Dams, the dam is considered to have no hazard from earthquakes provided static stability conditions are satisfactory and conventional safety margins exist.

6.3 Evaluation: There are no available soil testing records to verify the assumed soil parameters. Also, the dam has an upstream slope of 1V:3H which, based on the analyses, is not adequate according to the recommended design.

However, the visual inspection revealed no evidence of instability or stress of the embankment; therefore, the dam is considered stable and adequate for normal pool operations. But it is recommended that the upstream slope, according to state of the art practices, be checked for sudden drawdown conditions, assuming a minimum drawdown from normal pool.

## SECTION 7

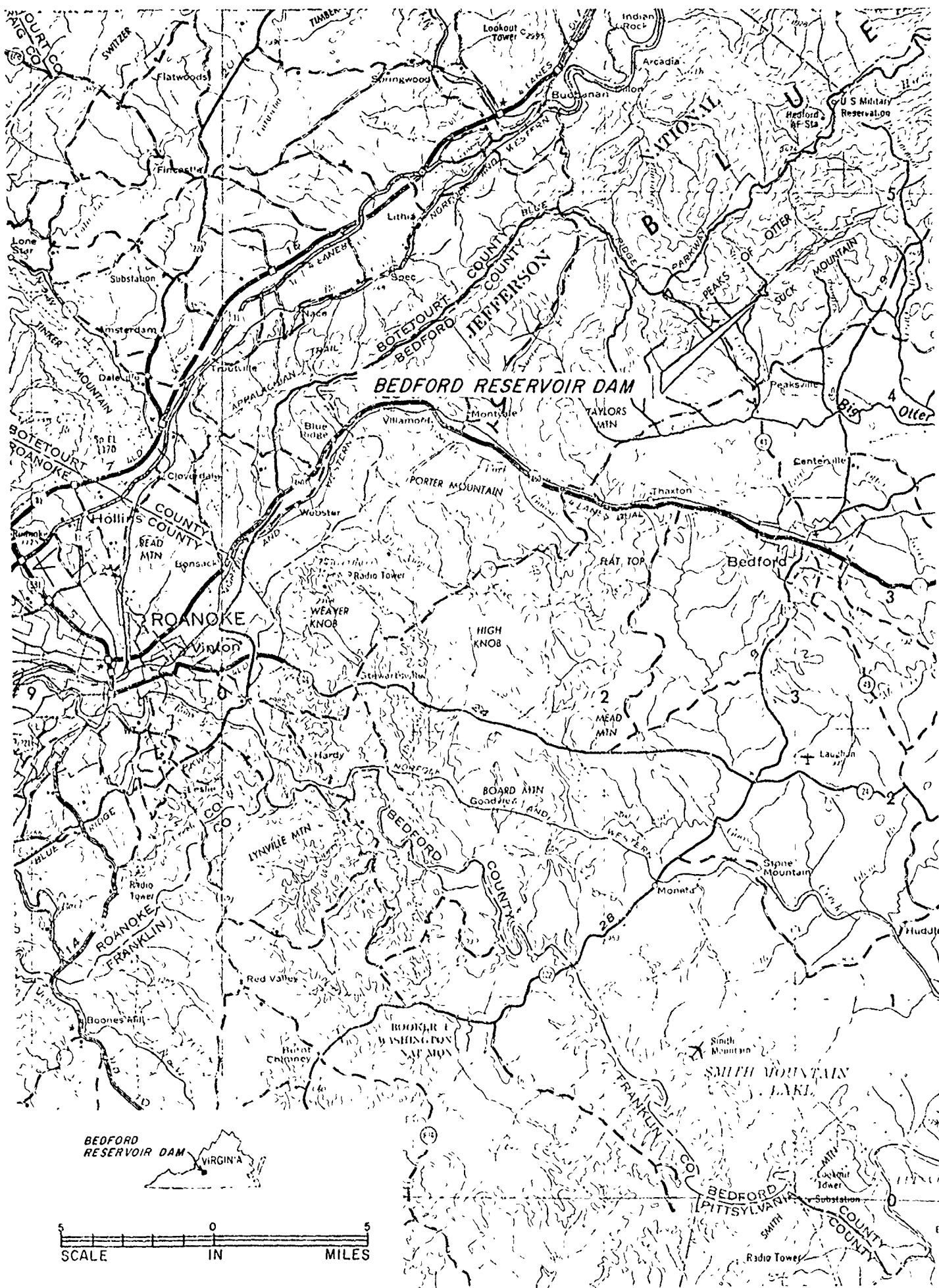
### ASSESSMENT, RECOMMENDATIONS/REMEDIAL MEASURES

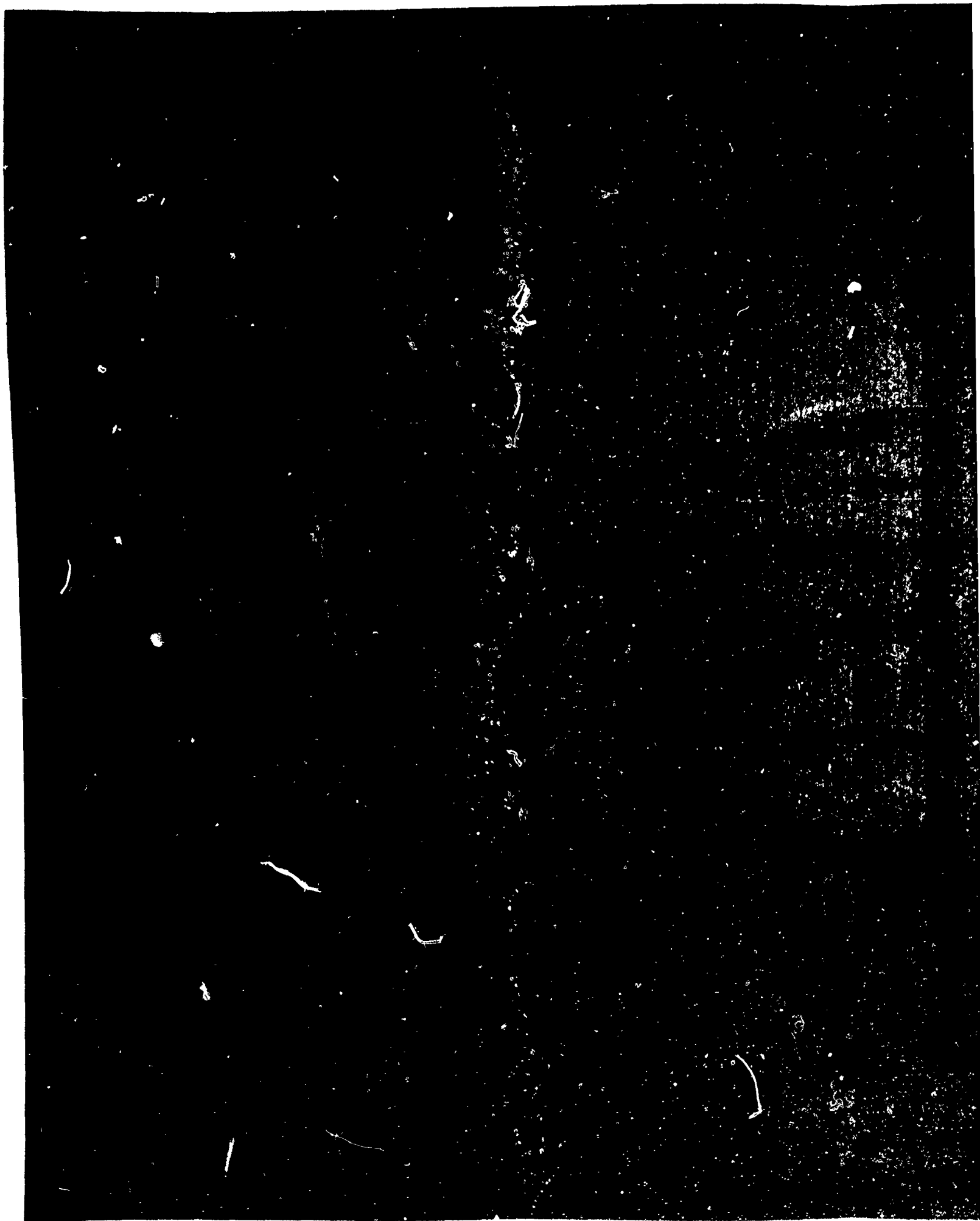
7.1 Assessment: The inspection revealed no findings that prove the dam to be unsound during normal pool operations. Available engineering data was adequate for review. The Spillway Design Flood (1/2 PMF) does not top the crest of the dam, therefore, the emergency spillway is considered adequate. The stability analyses indicate an adequate factor of safety on the downstream slope. But, the dam has an upstream slope of 1V:3H which, according to the given design calculations, has a factor of safety of 1.0 for the sudden drawdown condition. Overall, the dam is in adequate condition and will not require urgent remedial treatment.

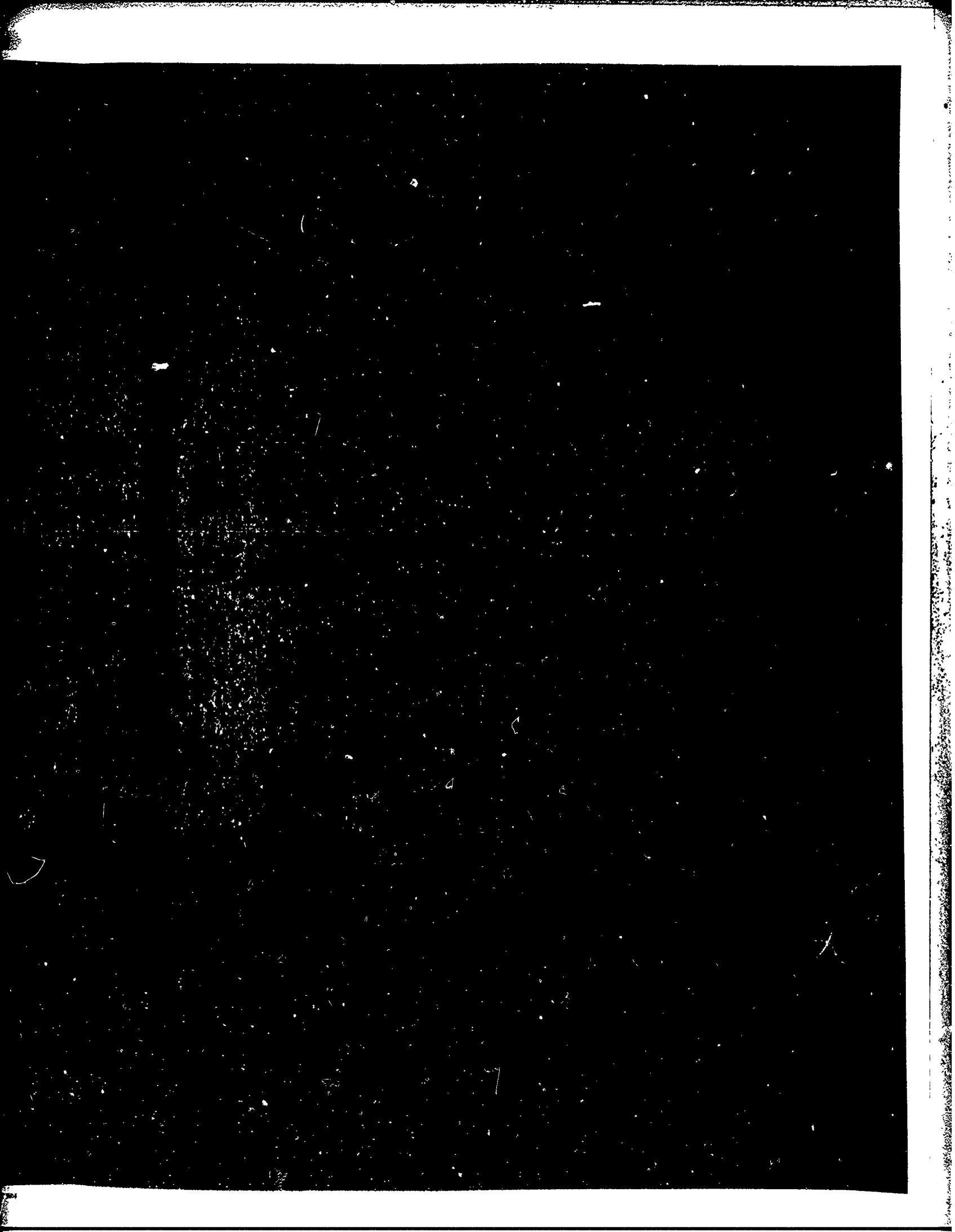
7.2 Recommended Remedial Measures: It is recommended within 12 months that stability analyses be performed on the upstream slope assuming a minimum drawdown from normal pool. It is also recommended that an annual maintenance program be initiated to maintain the integrity of the structure. The inspection revealed the following maintenance items that should be scheduled by the owner during a regular maintenance period:

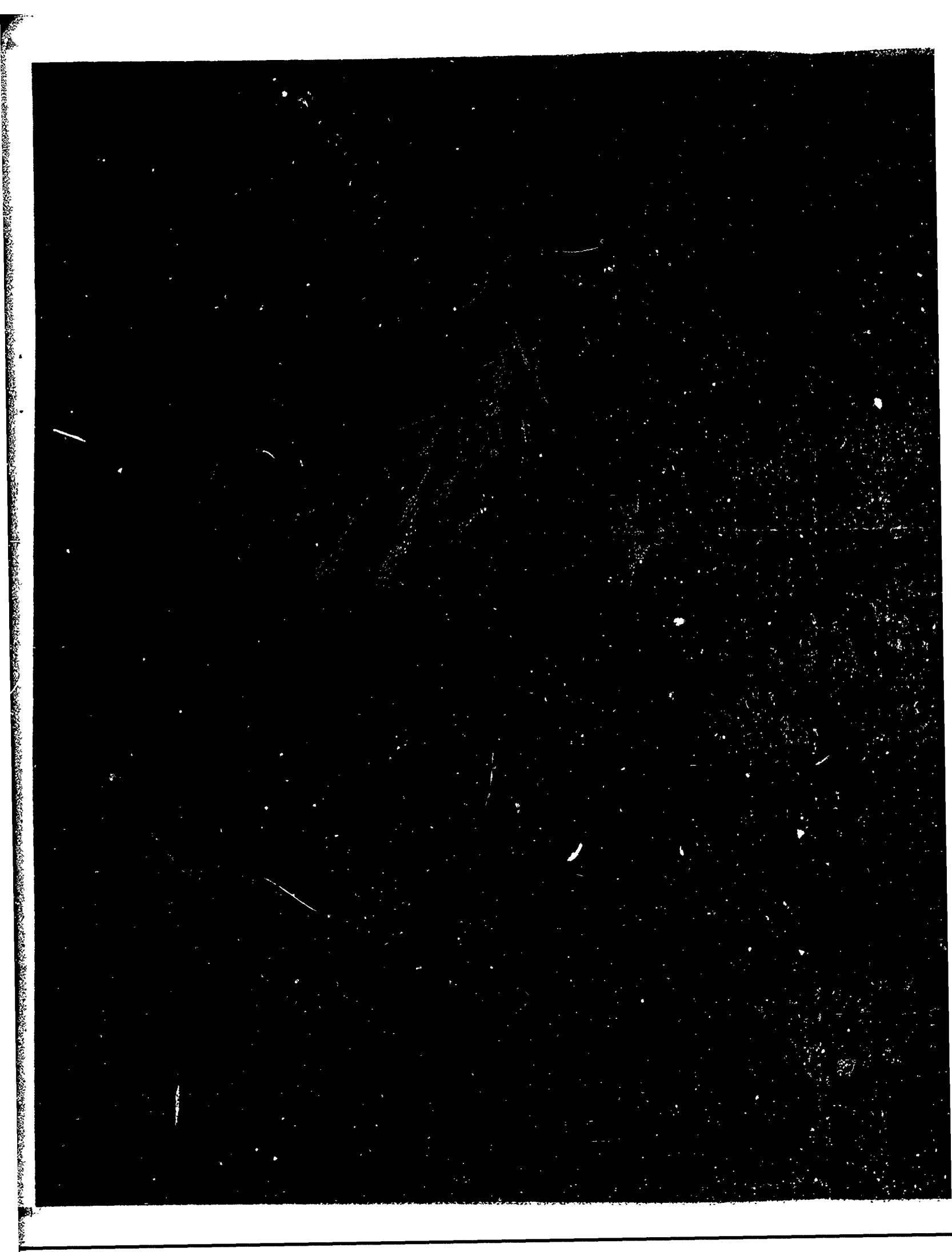
- a. Trim the embankment of brush, cut all trees at the trunk, and reseed the embankment.
- b. Clean out all the catch basins on the dam.
- c. Cut the tree growing in the emergency spillway.
- d. Riprap the eroded area in the channel downstream of the ungated spillway.
- e. Remove the vegetation and debris located on the right side of the downstream channel of the emergency spillway. Also, uncover the right subsurface drain and allow it to flow freely.

APPENDIX I  
MAPS AND DRAWINGS



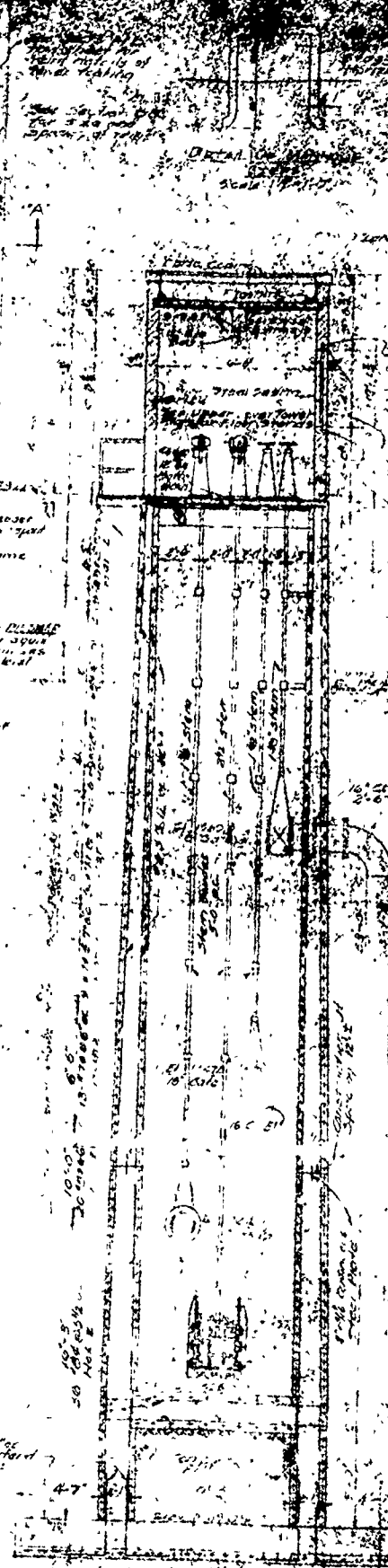
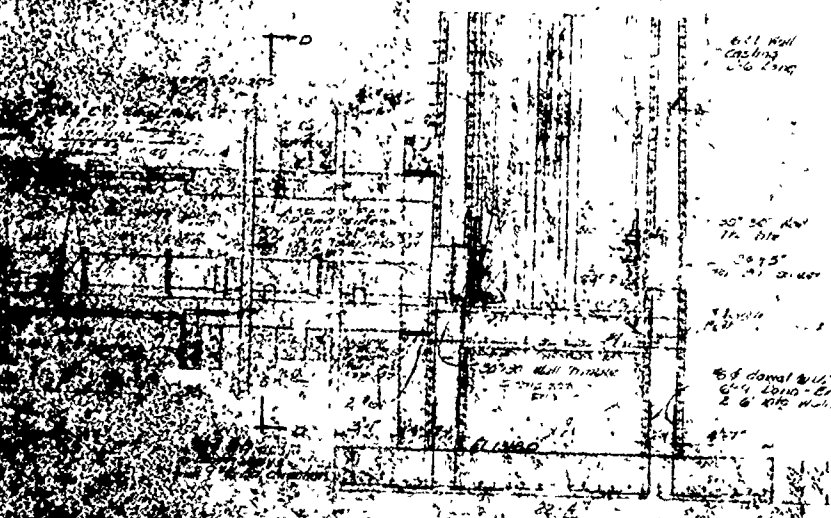
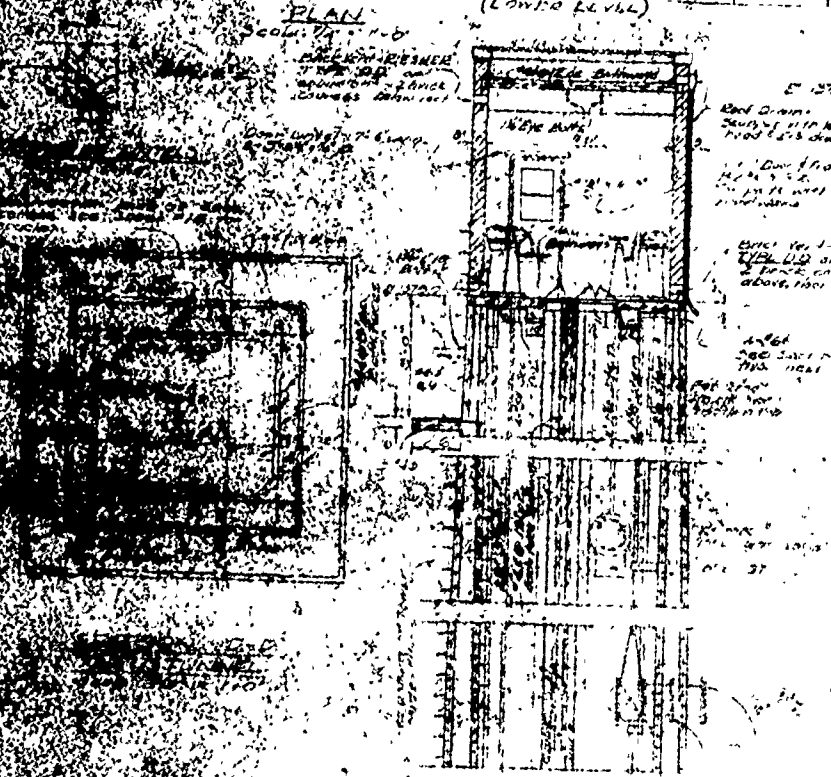
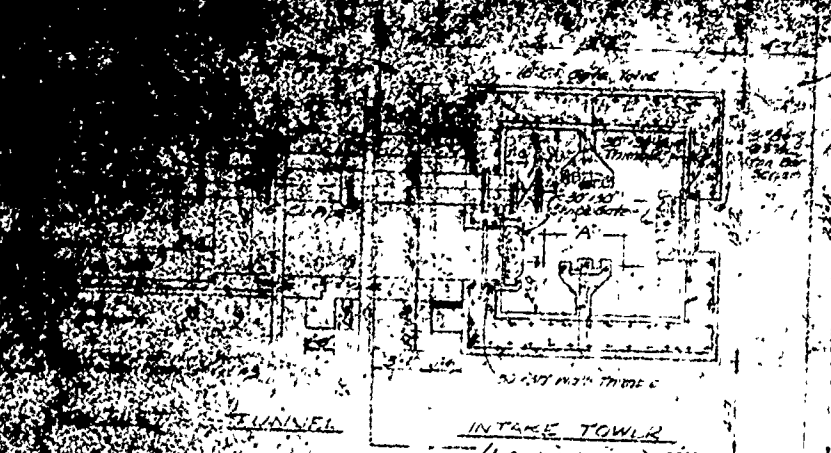


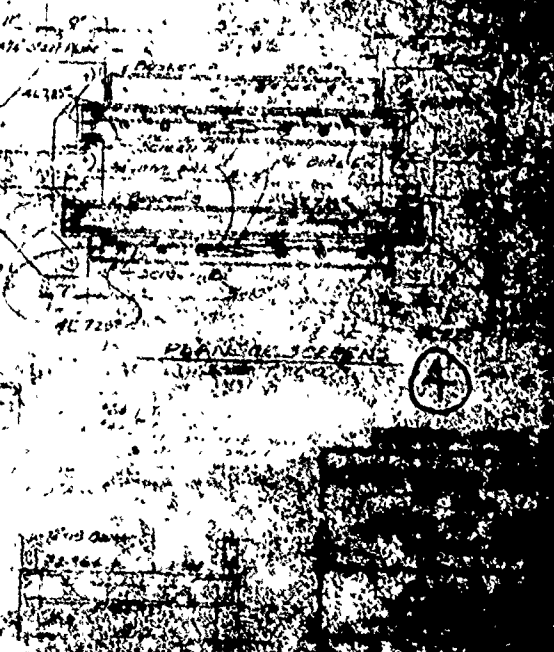
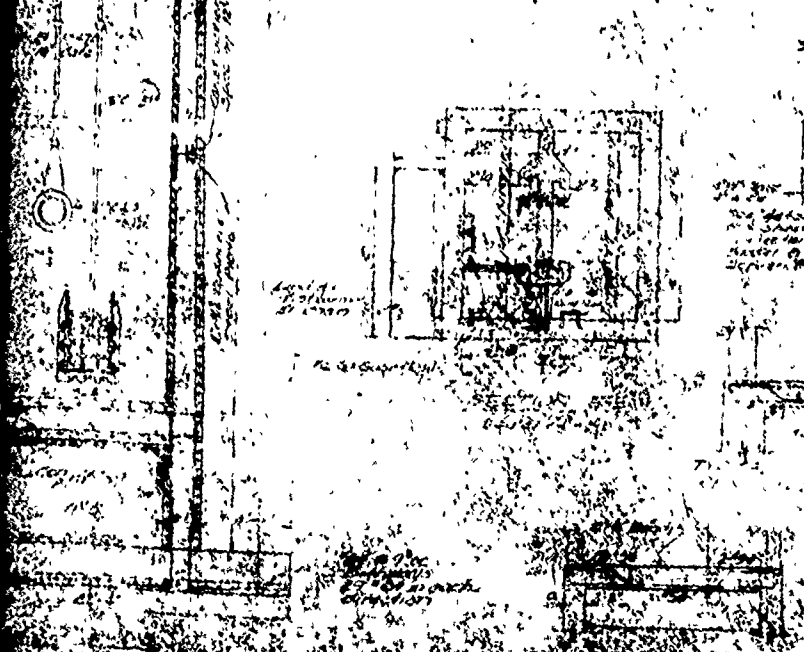
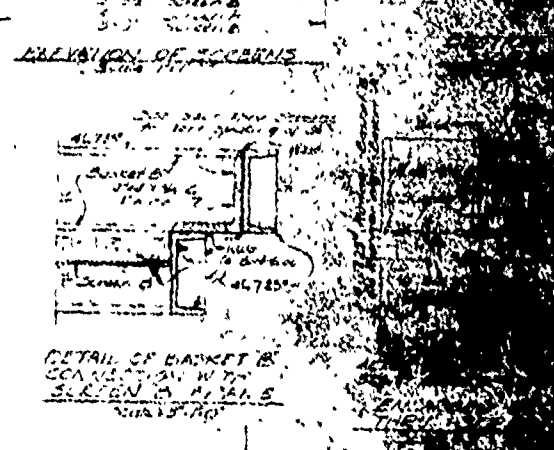
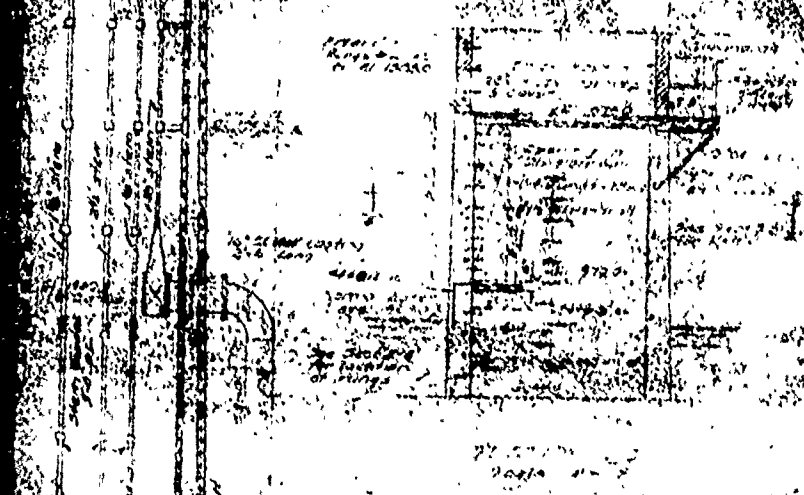
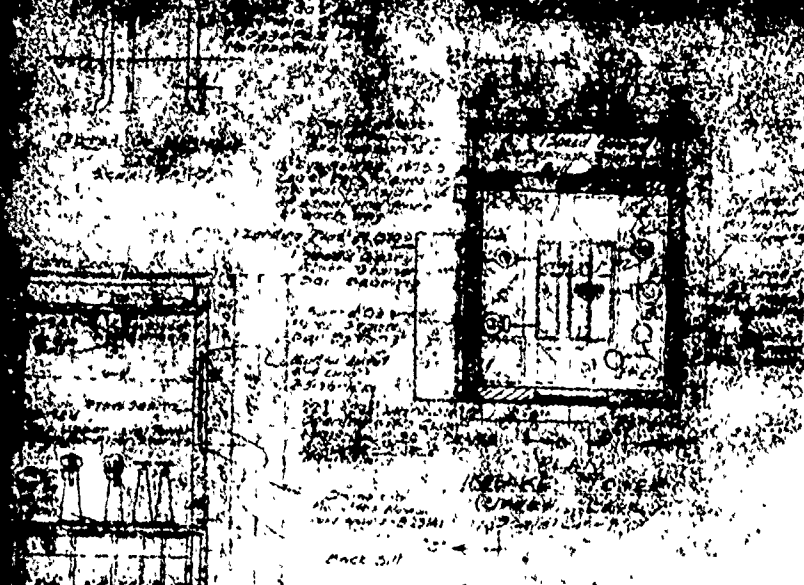


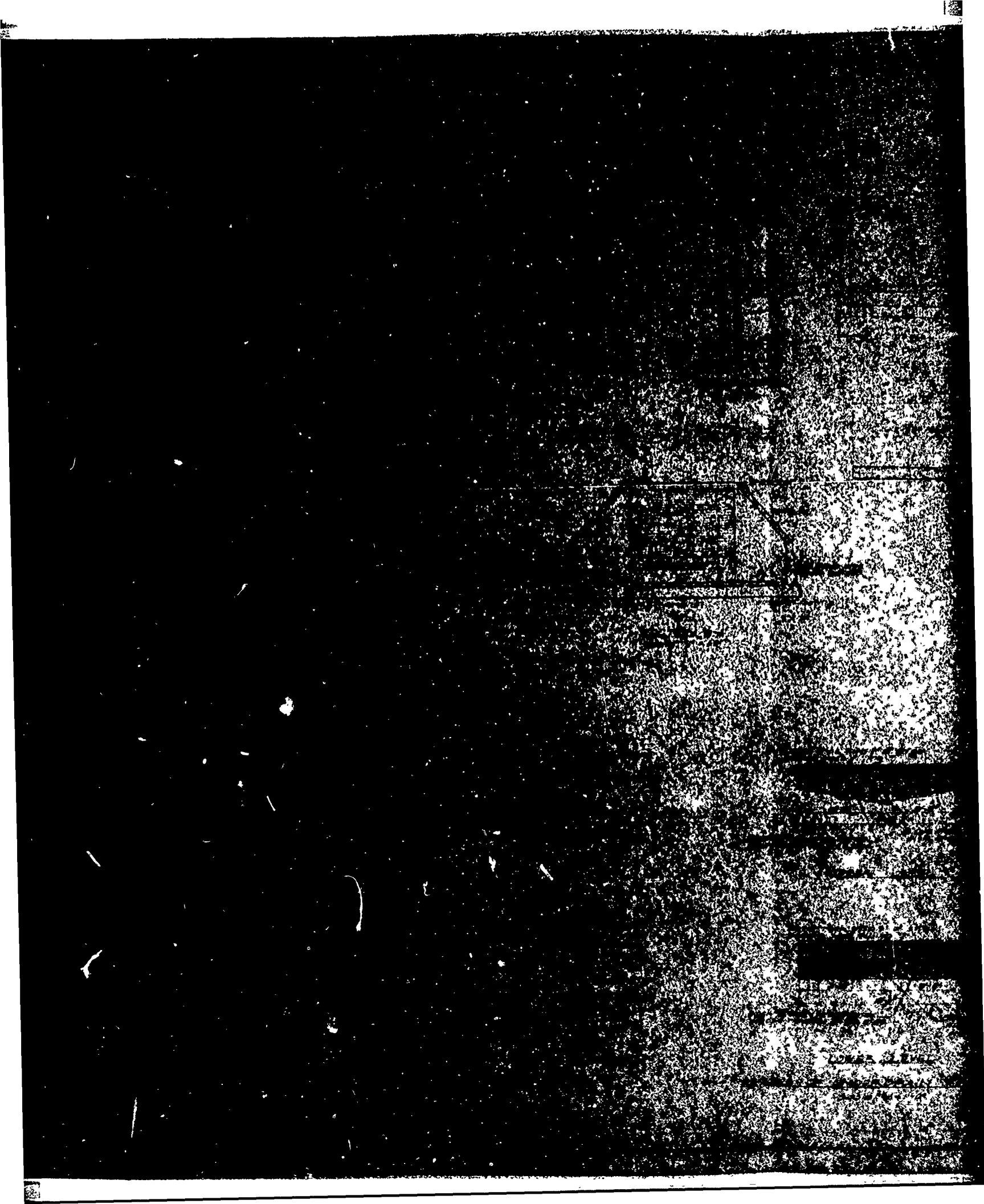




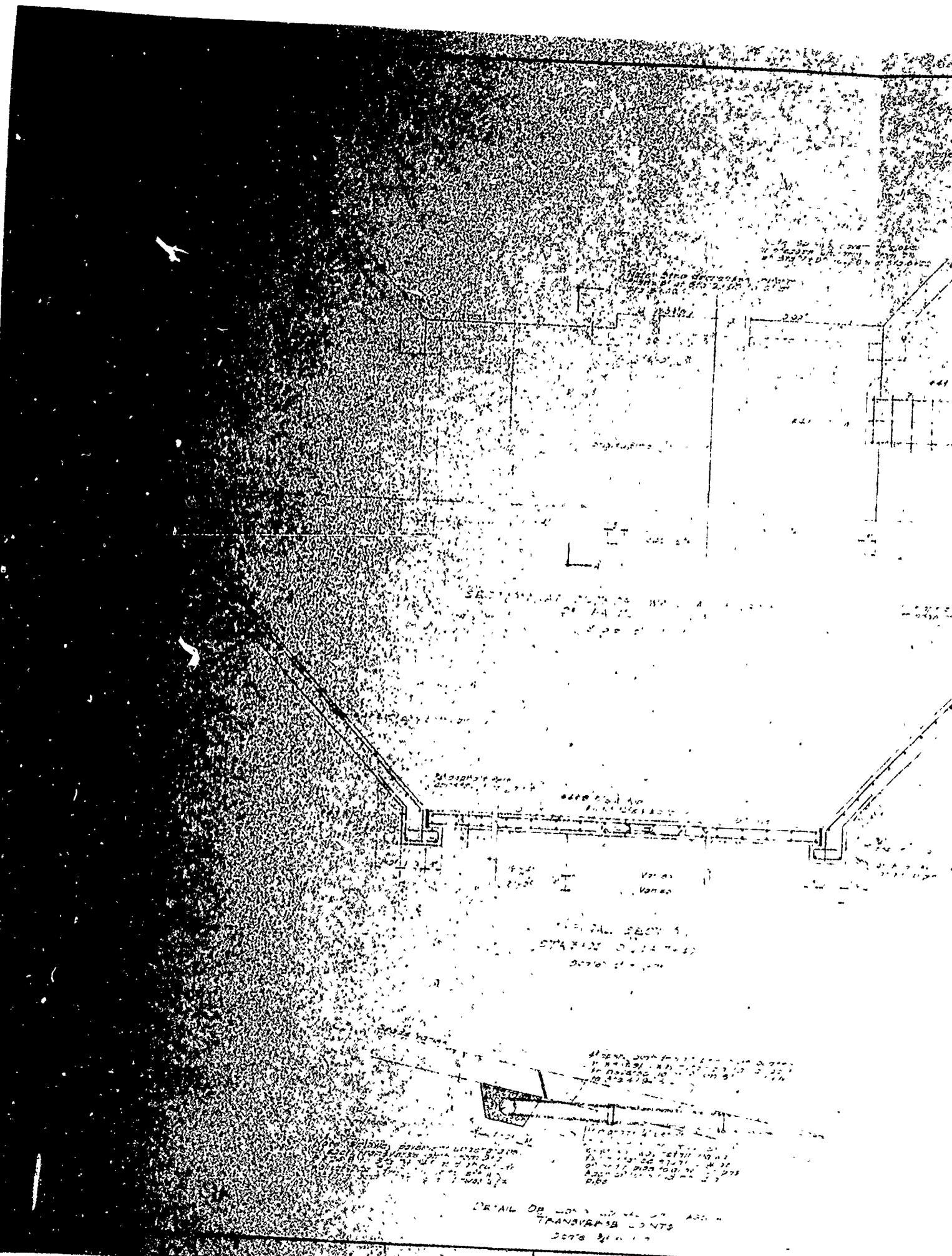










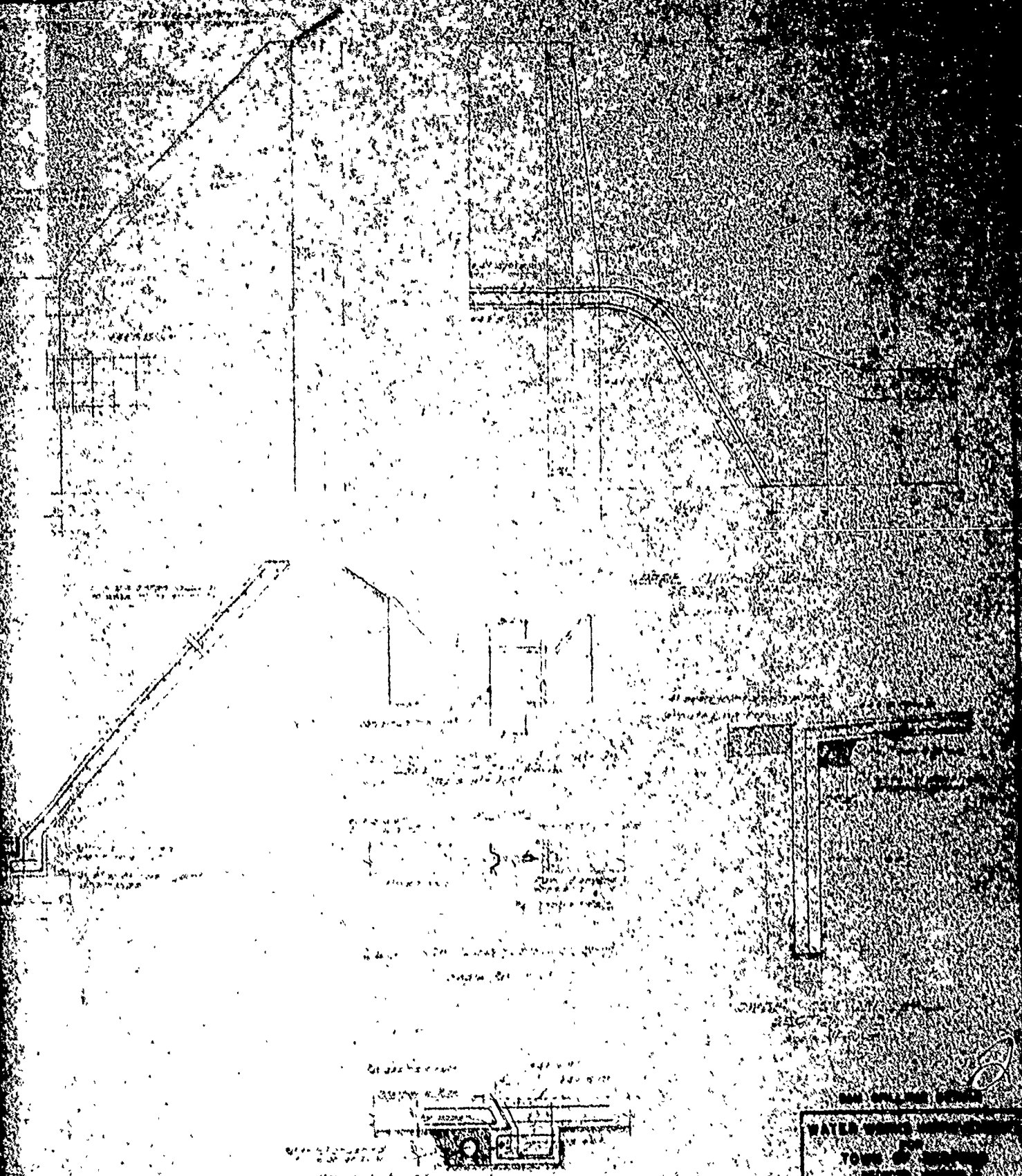


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SECTION 1  
STATION 0+14.75  
SECTION 1

DETAIL OF SECTION 1  
TRANSVERSE SECTIONS  
SECTION 1





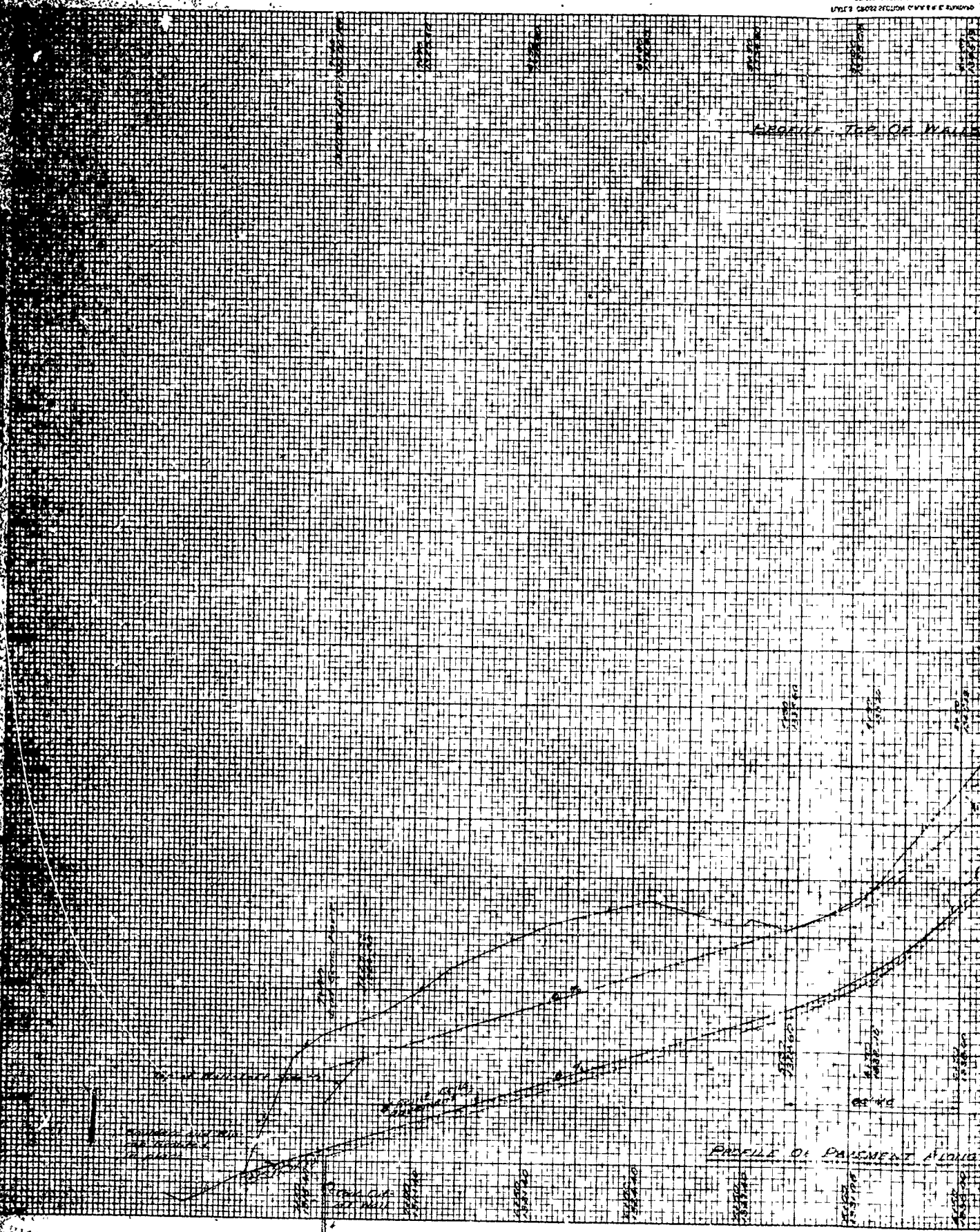
Very truly yours,  
 J. Edgar Hoover  
 Director

PROFILE - TOP OF WALL

PROFILE OF PAVEMENT ALONG

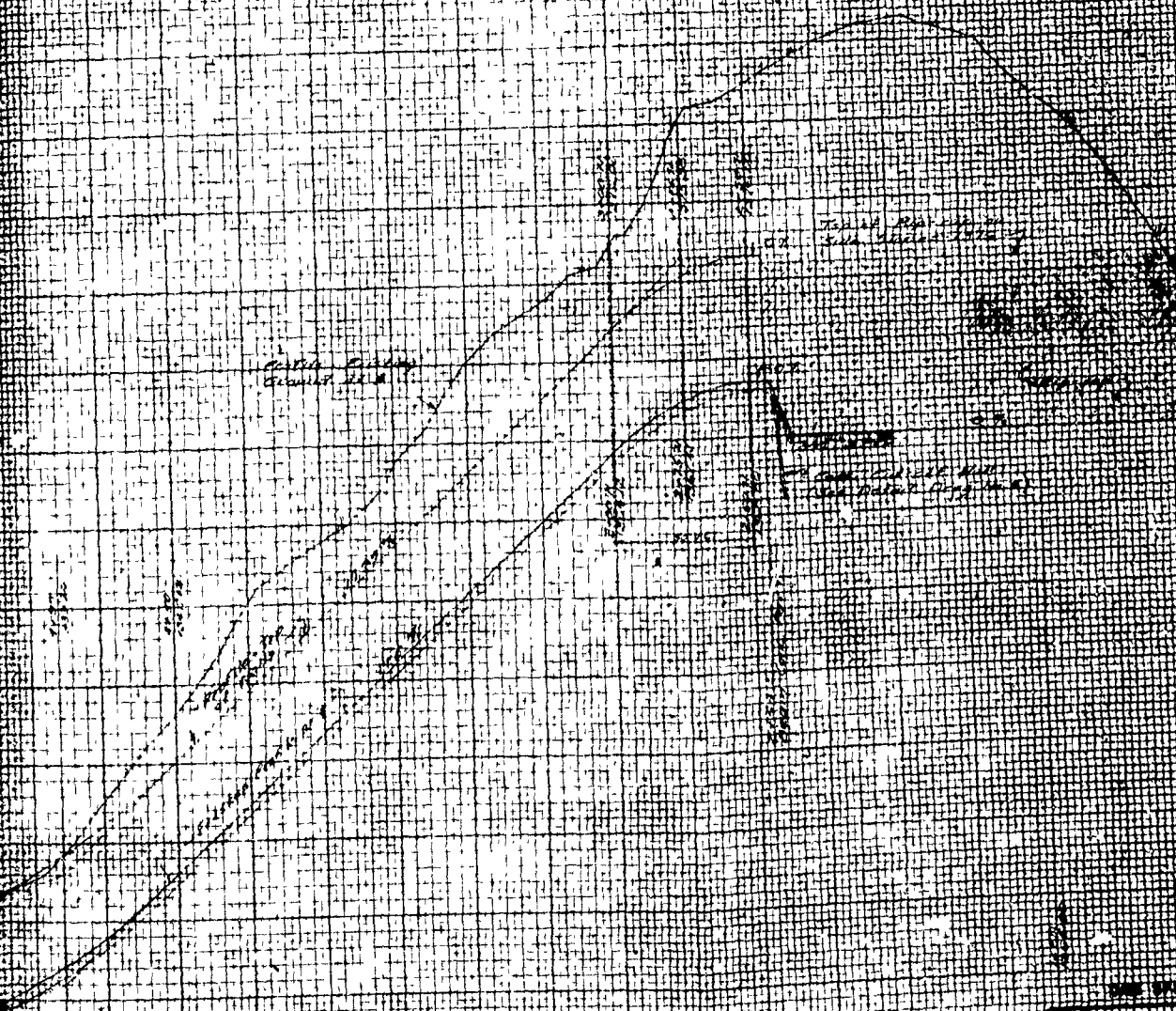
PROPOSED GRADE  
FOR PAVEMENT

PROPOSED GRADE  
FOR WALL



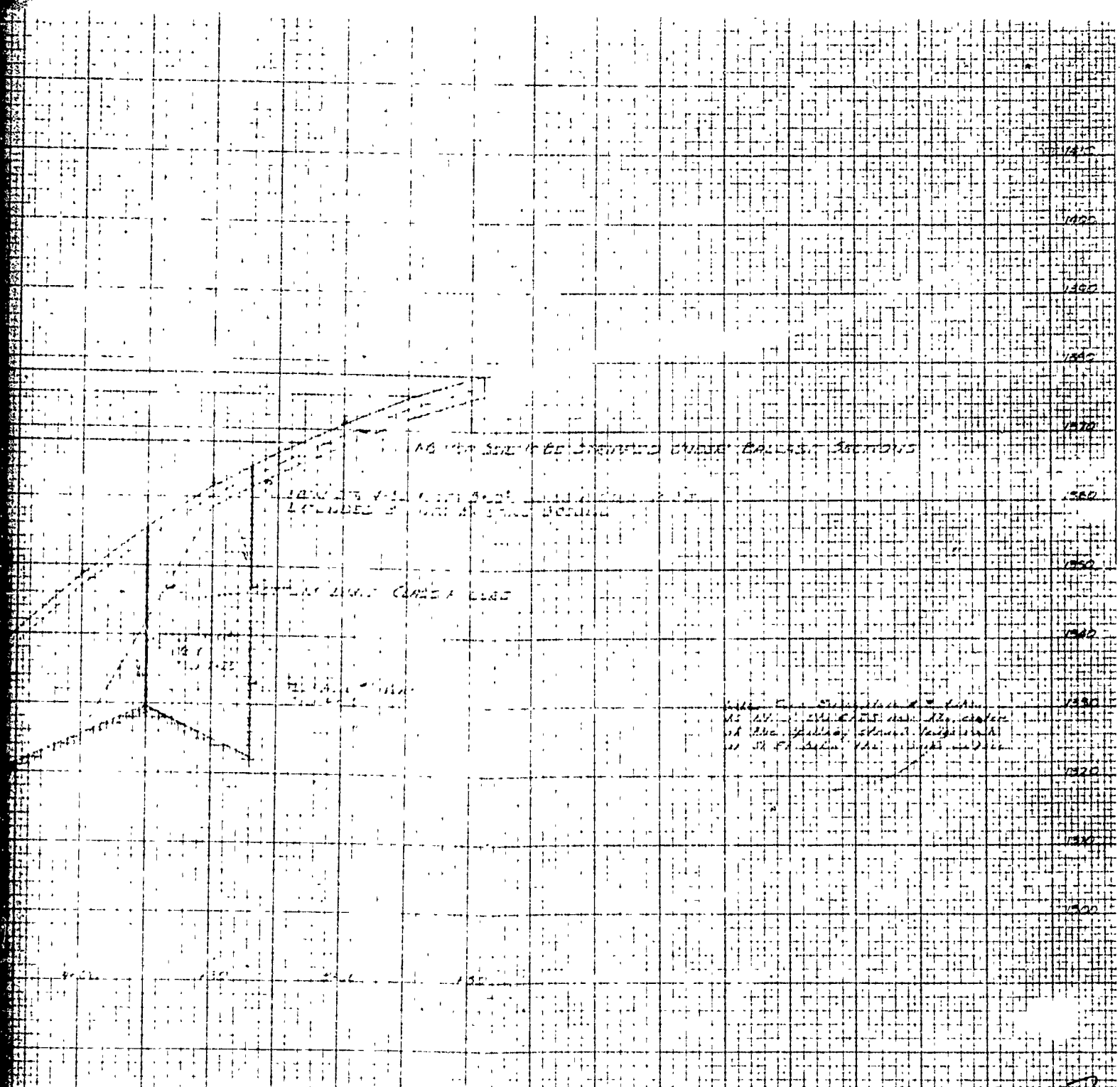


OF WALLS



PAVEMENT ALONG & OF SPILLWAY

NO. 2	WEBS CHECKED:	
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ELIMINATE	ROLLED	
	SHIPPED	
		BY
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AS PER SPEC. TO BE SHOWN UNDER BALLAST SECTIONS

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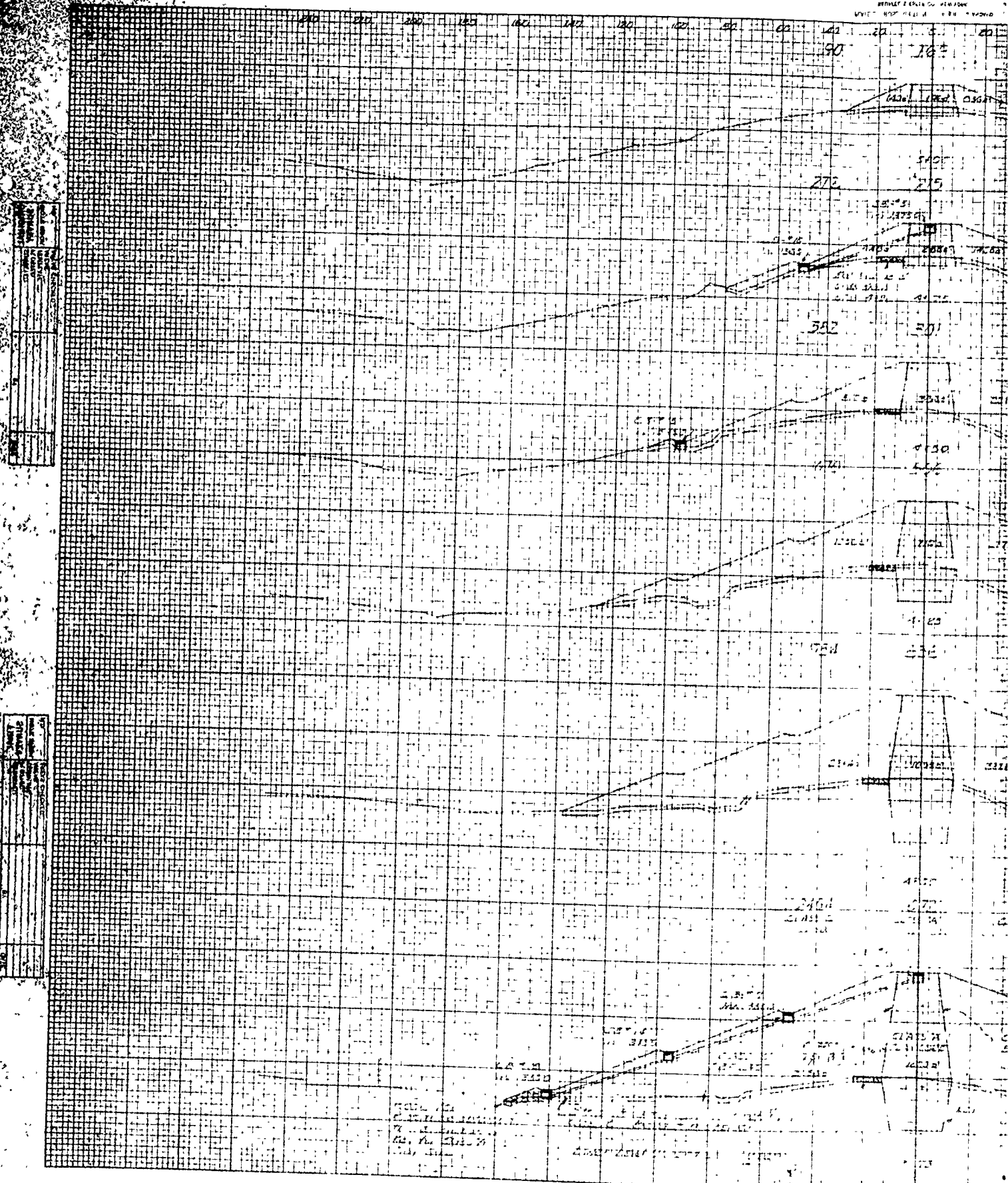
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PROFILE 6 DAM

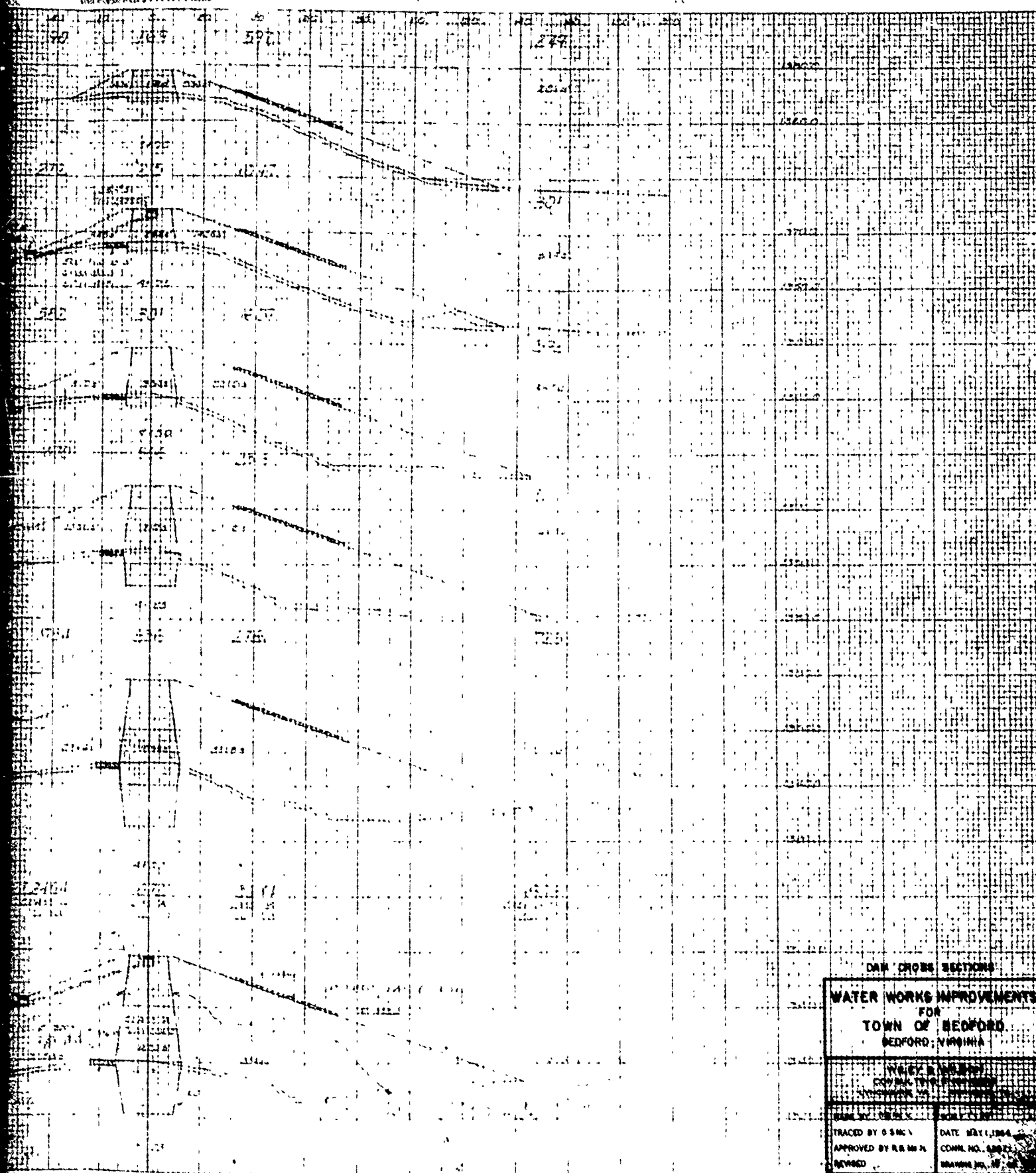
WATER WORKS IMPROVEMENTS  
FOR  
TOWN OF BEDFORD  
BEDFORD, VIRGINIA

PROJECT NO. 54-245	
DATE: 10/1/54	
TRACED BY: G.S.M.V.	DATE: 10/1/54
APPROVED BY: G.S.M.V.	DATE: 10/1/54
REVISOR:	DATE: 10/1/54
DRAWING NO. 54-245	

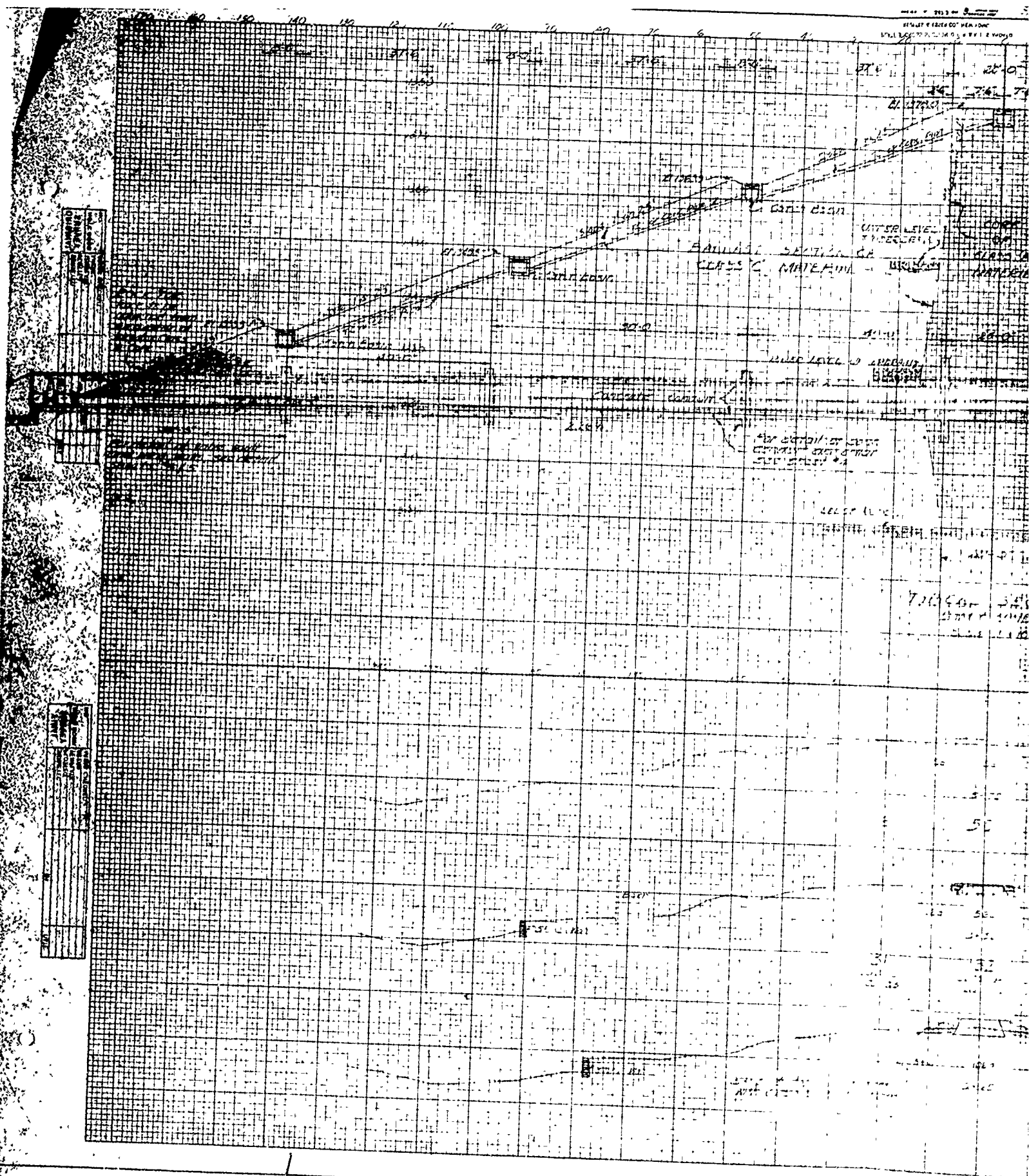
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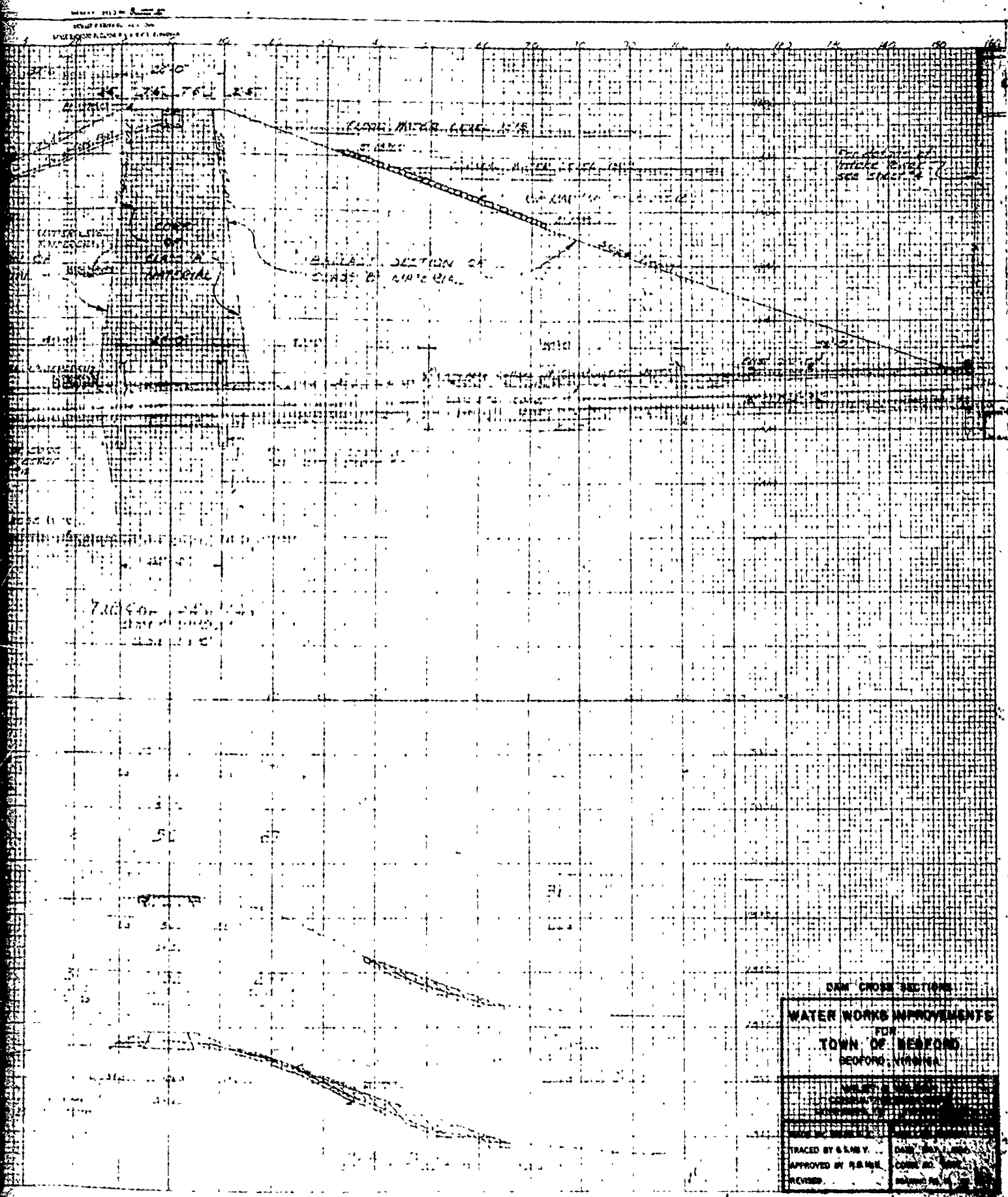






DAN CROSS SECTIONS	
WATER WORKS IMPROVEMENTS FOR TOWN OF BEDFORD BEDFORD, VIRGINIA	
WILEY R. BERRY CIVIL ENGINEER BEDFORD, VA.	
DESIGNED BY W. R. B.	CHECKED BY W. R. B.
TRACED BY G. S. M.	DATE MAY 1, 1964
APPROVED BY R. B. M.	COMM. NO. 6807
REVISED	DRAWING NO. 10-64



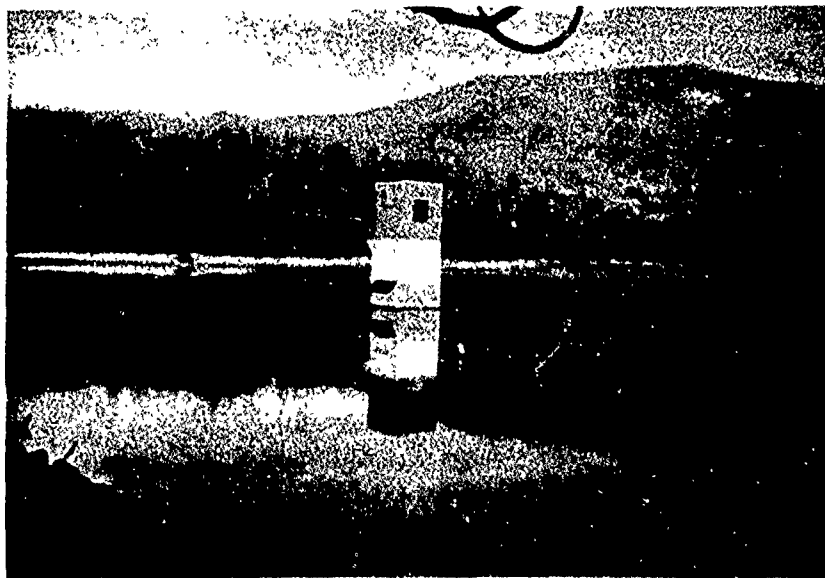


DAM CROSS SECTION	
WATER WORKS IMPROVEMENTS	
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TOWN OF BEDFORD	
BEDFORD, VIRGINIA	
DESIGNED BY	
CHECKED BY	
APPROVED BY	
REVISION	
TRACED BY S. H. V.	DAM, TOWN OF BEDFORD
APPROVED BY S. H. V.	CORR. BY
REVISION	REVISION

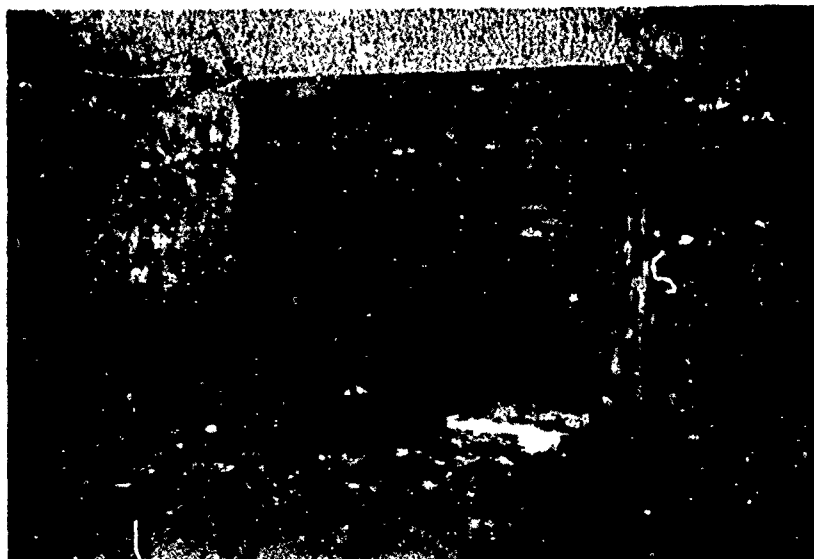
APPENDIX II

PHOTOGRAPHS





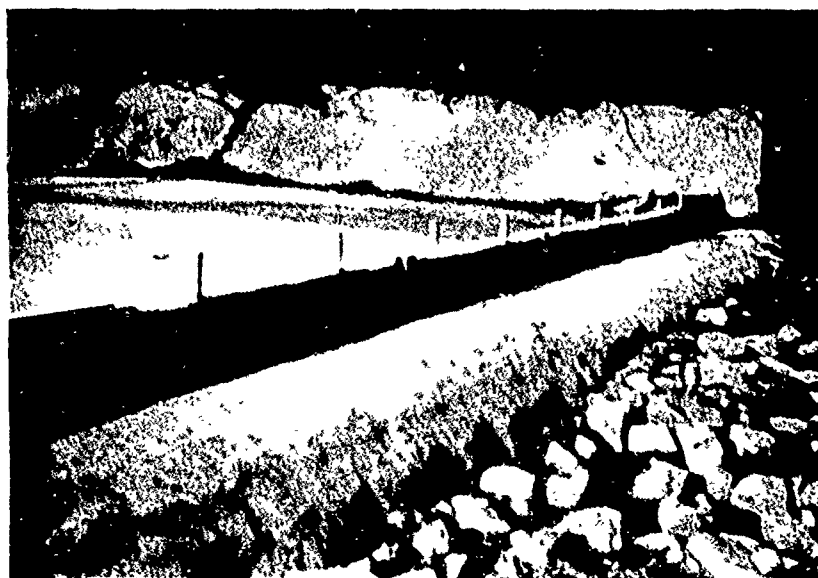
INTAKE STRUCTURE



OUTLET STRUCTURE



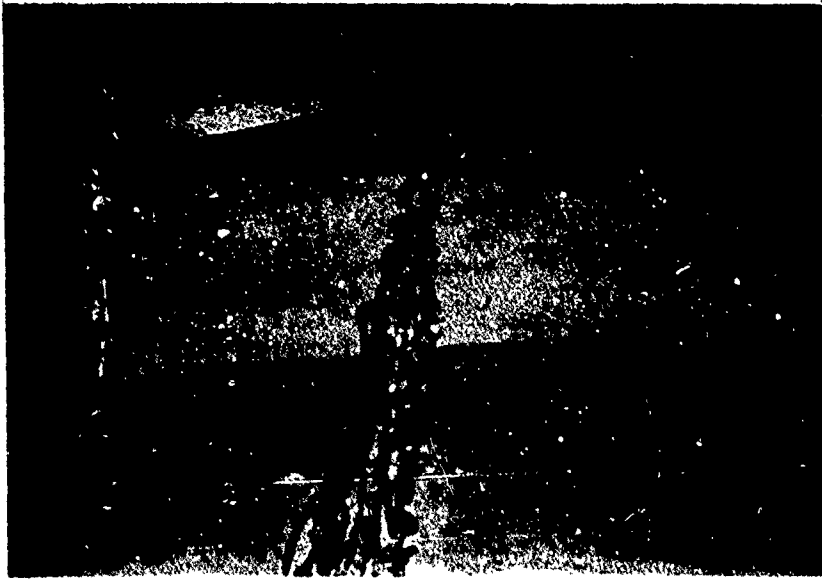
TOP OF DAM-THICK LOCUST GROWTH



CREST OF SPILLWAY-MISSING FLASHBOARDS



ERODED CONCRETE AND RIPRAP FAILURE DISCHARGE CHANNEL



CONSTRUCTION JOINT IN SPILLWAY-FILLED WITH COLD POUR



SOME VEGETATION GROWING IN LEFT WING WALL

APPENDIX III  
FIELD OBSERVATIONS

### APPENDIX III

#### PHASE I - FIELD OBSERVATIONS

Name of Dam: Bedford Reservoir Dam (VA 01904)  
County: Bedford State: Virginia  
Coordinates: Lat. 3726.6 Long. 7933.0  
Date of Inspection: 1 December 1978  
Weather: Fair  
Temperature: 45° F  
Pool Elevation at Time of Inspection: 1362.75 m.s.l.  
Tailwater at Time of Inspection: 1316+ m.s.l.

#### Norfolk District Inspection Personnel:

Jim Robinson, Hydrology  
Dave Pezza, Soils  
Richard Klein, Trainee

#### Other attendees:

Neil Obershain, State Water Control Board  
Walter Kent, City of Bedford  
Everett Wood, City of Bedford  
Terry Dooley, City of Bedford

1. GENERAL: At the time of inspection, the level of the water in the reservoir was 6.25 feet below the normal pool level of elevation 1369.0.

#### 2. EMBANKMENT

2.1 Surface Cracks: The slopes, crest, and abutment contacts were inspected and no cracks were found. However, the face of the dam was covered with thick vegetation inhibiting a thorough investigation. The vegetation consisted of brush and locust trees up to 2 inches in diameter.

2.2 Unusual Movement: No unusual movement was noted on the dam. The vegetation inhibited observations.

2.3 Sloughing or Erosion: No sloughing or erosion was noted.

2.4 Alignment: The vertical and horizontal alignment of the dam did not deviate from the as-built drawings.

2.5 Riprap: No riprap failures were found on the upstream face of the dam.

2.6 Junctions: No problems were observed at the junctions of the embankment and left abutment, and spillway and dam.

2.7 Seepage: No seepage was observed.

2.8 Drains:

2.8.1 Surface Drains: Several of the drains were plugged with debris.

2.8.2 Foundation Drains: The drains were not observable during the investigation. It was noted from the drawings that the blanket drains drain into the surface drainage system. It is not clear where the toe drain empties. At the time of the inspection, the 12-inch and 15-inch pipes discharging into the outlet works were submerged. The 12-inch pipe appeared to be flowing full and flow was evident from the 15-inch pipe.

3. Water Works:

3.1 Intake Structure: There is no access to the intake structure from the dam. The structure is concrete and showed no visible signs of deterioration. An inclosed structure sits on top the intake prohibiting observation of shafts used to operate sluice gates. No determinations of the intake structure could be made during the inspection. State Police divers found no problems during inspection in the fall of 1977.

3.2 Outlet Works: A 30-inch by 30-inch sluice gate serves as the principal spillway by connecting the intake and the discharge tunnel. The gate is opened partially allowing flow downstream. The tunnel dimensions are 6 feet wide and 5 feet high with a 30-inch by 30-inch opening in a bricked wall at the discharge end. Flow through a 16-inch water supply pipe could not be determined. A wooden plug was missing from 16-inch outlet. There was a 2-inch coating of iron bacteria in the outlet and downstream channel.

3.3 Outlet Channel: A discharge channel extends from the toe of the dam to approximately 20 feet downstream. Wing walls and large boulders line the channel which is approximately 3 feet deep near center of channel just below the exit channel of the discharge tunnel. The tailwater was approximately 0.5 feet below the tunnel discharge invert. A large quantity of iron bacteria was observed in the discharge tunnel and in the downstream channel area.

4. Emergency Spillway: The spillway is a concrete side-channel located to the right of the earthen embankment. Flashboards were added in 1966 and concrete reinforcement for the flashboards in 1975, which raised the normal pool elevation to 1369.0. Portions of the flashboards are missing near the right abutment. The concrete discharge channel appeared in good condition.

No spalling and scaling were evident and the expansion joints are sealed with bituminous asphalt or cold pour. Several of the asphalt sealed joints have light vegetation growing in them. There are several 2.5 inch diameter hydrostatic relief plugs in the channel. Alligator cracking was noted in the concrete slab at station 7+00 with patching apparent. A tree is growing out of a joint in the left channel wall at about Station 2 + 90. The drain discharging into the channel at about Station 4 + 85 was flowing freely with clear water. A pile of soil is resting in the right wall at about Station 5 + 25. The left wall subsurface drain was flowing freely with clear water. The right drain was not found. The riprapped downstream channel has eroded away on the left side. The right side is overgrown with heavy vegetation and debris has blocked the right drain.

5. Reservoir: The area surrounding the upstream reservoir consists of mild to moderate sloping wooded mountain terrain. No observations of sediment could be made. The water was clear and free of debris.

6. Downstream Channel: The channel immediately beyond the outlet had mild slopes up to 12 feet in height, then steep slopes to the left and no change in slope to the right. No erosion was observed in the downstream area. No homes are in the apparent flood plain.

7. Instrumentation: A staff gage was located on the intake structure to allow visual readings of elevation variances from normal pool elevation.



**APPENDIX IV**

**PRELIMINARY MEMORANDUM ON GEOLOGICAL CONDITIONS**

BYRON N. COOPER  
GEOLOGIST

BOX 634  
BLACKSBURG, VIRGINIA

V. P. I. OFFICE  
6261 EXT. 360

RESIDENCE  
100 COUNTRY CLUB DRIVE  
6736

February 23, 1953

PRELIMINARY MEMORANDUM ON GEOLOGICAL CONDITIONS AT A  
PROPOSED DAM SITE ON STONY CREEK, NEAR BEDFORD, VA.

On Thursday, February 19, 1953, the writer in company with Mr. R. W. Martin, Mr. McNutt, and Mr. Dodl made a preliminary study of possibilities for impounding surface water for a new source of supply for the Town of Bedford. The area investigated lies in the valley of Stony Creek a short distance upstream from the present small reservoir. Two sites were discussed in the field: one dam site is just upstream from the northern boundary of a tract owned by the Town and is situated on the Cross land; another dam site being considered would cross the valley at a point above a small cemetery situated along the west side of the valley bottom. These sites will be referred to, respectively, as the lower and upper sites.

The upper site.— Of the two locations considered, the upper site is definitely the less desirable one. A dam across the valley at this location would have to be about 600 feet long, or about twice the length required at the lower location. At the upper location, the possibilities for a suitable spillway and for convenient anchoring of the structure against the valley walls are poor. The major objection to the upper site is that its utilization would entail costly excavation down through the alluviated floor of the valley over a length of at least 600 feet,

in a location where there is every indication of a thick and continuous alluvial cover all the way across the valley. This material would, of course, have to be completely penetrated and cut off by a trench down into the bedrock, in order to insure against leakage from the proposed reservoir. The considerable excavation that would be entailed by this procedure would be costly.

The lower site.— The bedrock at the lower site is a massive dark-colored greenish-gray rock composed largely of hornblende and plagioclase feldspar, which has been called Lynchburg gneiss. The rock is sufficiently basic to weather down to a bright reddish-brown soil which is relatively dense, plastic, and water-tight. Exposures along the east side of the Stony Creek road indicate two major joint patterns in the bedrock. One pattern or system follows the rift of the rock and strikes N.  $50^{\circ}$  E. with a dip of  $55-75^{\circ}$  SE. The other system of regular fractures strikes about N.  $30^{\circ}$  W. and is nearly vertical. The second system is not as well developed as the first and joints in the minor system are offset by the rift joints. There is some minor evidence of horizontal or sheeting joints in the rock in one exposure upstream from the lower site. The rocky character of the east valley wall indicates beyond any question that bedrock is practically at the surface. The spur just west of Stony Creek at the lower site does not appear to be a hill composed of gravelly alluvium or loose blocks of alderock. The surface indications are plain that this hill has a core of massive gneiss like that which is well exposed along the east wall of the valley. The soil on this spur is definitely residual and must grade down within a few feet into gneiss. This condition is of major importance in evaluating this particular site for a dam, because the spur offers not only a suitable place for a spillway but also serves to

but remained to be so deeply weathered---and this in an area where so great topographic relief would be expected to so accelerate the flow of streams as to flush away all loose rock.

If the spur in which the west end of the dam is to be founded were an aggregation of fresh boulders, it would be a most unsuitable anchor for a dam, because the interstices between the boulders would be unfilled with impervious clay. Such clay as does occur in the interstices between the weathered boulders of the mantle composing this spur could only have been formed where it now is. Indeed, the clay exhibits much of the texture of the crystalline rocks from which it was formed by chemical weathering. The conversion of the abundant calcic feldspar in the boulders during weathering into clay has served to plug the voids that surely must have existed when the boulder deposit was fresh.

For all practical purposes, the spur to be used as an anchor for the west end of the dam is just as suitable a tie-point for the proposed structure as if it were a normally derived, truly residual mass formed from the chemical decay of the indigenous bedrock. This is one of the most important and critical conclusions drawn from the borings.

The cobble-stream flat stretching from the toe of the spur described above over to the road along the creek was shown by Hole 2 and by subsequent excavations made with a "clam-shell" to be a neatly imbricated mass of river cobbles attaining a maximum thickness of about 27 feet. The bore hole and the excavation made with the "clam-shell" show that the boulders rest upon fresh bedrock with the deepest or thickest point of the boulder deposit being at the approximate locus of Hole 2. The high permeability and coarseness of the boulder deposit means that the excavation for the dam must be carried down through the gravel-cobble deposit and into bedrock. The neat imbrication of the river stones below creek level will facilitate excavation of the material and permit retention of a fairly well defined trench or cut-off ditch across the deposit to be used for back-filling with clay of Class A grade.

The thick river deposit at the dam site is another anomalous aspect of the dam site and reservoir area. In a region of so rugged relief, it would be expected that the surface streams would be actively eroding the bedrock. Not so with Stony Creek. In former times, probably when the climate was colder and there was more precipitation in its watershed, Stony Creek excavated a channel in the bedrock, but later with diminution in flow and lessening of its transporting capacity deposited in its old channel the deposit of boulders and cobbles encountered at Hole 2 and in the "clam-shell" excavations. Probably, when the present type of climate started to prevail, the deposition of boulders in the old, deeper channel of the stream was rapidly effected.

To summarize, rigorous mechanical weathering and mass-wastage of the crystalline rocks on the Peaks of Otter, which possibly existed during some early part of the Great Ice Age, created a great colluvial deposit at the base of the mountains which gradually crowded Stony Creek

farther and farther southeastward until the stream became wedged against the base of Suck Mountain. In this position, the stream gradually undercut Suck Mountain, oversteepened the northwest slopes of that mountain, and thereby caused the ledge-rock to slide in large chunks or pieces down into the creek bed. These pieces, larger than the stream could possibly move, may have served as baffles promoting deposition of the boulders composing the alluvial deposit below stream level. The enormous mass of slide rock off of the peaks of Otter has undergone thorough chemical weathering and has transformed the once permeable mass, or at least the part involved in the proposed reservoir and dam, into an impervious and mechanically stable fill which will serve effectively as a good and safe tie point for the proposed structure.

#### Summary of Core Hole Data

##### Holes drilled into bedrock wall on Suck Mountain side of valley.-

Holes 4 and 5 found tight and relatively joint-free impermeable bedrock at hole depths of 7 and 8 feet respectively. For Hole 5, this means that back into the face of Suck Mountain lateral excavations to firm and tight bedrock will have to be carried only 8 feet. Hole 4 indicates that the same type of bedrock will be encountered at an elevation of about 5 feet below road level. The east end of the proposed dam, therefore, will be assured of a good rock anchor which will be too tight to allow appreciable piping of water around that end of the impounding structure.

Hole 6 drilled on a 15-foot monolithic slab of rock, presumably derived from a slide off Suck Mountain, penetrated the boulder at a depth of about 5½ feet to encounter river cobbles down to about 25 feet where bedrock begins. Hole 2 shows 27 feet of river cobbles resting on fresh bedrock.

Hole 8 near the eastern toe of the spur in which the west end of the dam will be founded shows tight clay down to 23 feet. This clay includes some soft crumbly residual boulders. The undisturbed material down to 23½ feet will be reasonably water-tight, but the lower 2½ feet down to where bedrock begins at 26 feet may represent a feather edge of the boulder deposit underlying the valley flat. Excavation down to bedrock will be necessary in order to explore this thin permeable zone and to cut it off with a back-fill of Class A clay.

Holes 1, 3, and 7 higher on the spur show the thickness of weathered colluvial material to be as great as 51 feet. Dry samples of the soil from Holes 7 and 8 indicate impermeable material resulting from the thorough chemical decay of the originally colluvial deposit.

Cross section accompanying this report indicates the types of material occurring along the center line and the probable contour of the fresh bedrock below the weathered boulder deposit and below the stream fill.

#### Borrow Area For Class A and B Material

Investigation of the clay in the small open ravine to the west of the dam was done by hand auger. Thicknesses of fairly heavy, compactible dense clay appear to range up to about 11 or 12 feet, with average thickness of about 7 feet. A representative sample of this clay, probably the best grade present in the proposed borrow area, was collected and sent to Froehling and Robertson, Inc., for soil analysis. The borrow area can be expected to show some boulders, and probably some exploratory digging with a power shovel or scoops will be necessary to determine where a large enough section can be worked as a unit to allow mechanized panning of the material. It should be anticipated that the area of borrow will have to range up to 1800 feet away from the dam. A number of pits will have to be used, certainly including some in the old orchard area just southwest of the main area where material will be borrowed.

In general, it can be expected that the best grade of material will directly underlie the top soil, with Class B material under the top 3 or 4 feet of A material.

#### Spillway Section of Dam

The borings done near the proposed location of the spillway indicate that the spillway will have to be founded on weathered boulder clay rather than on bedrock. The clayey nature of the material will require consideration in designing the spillway apron, and areas below the concrete slabs of the spillway will have to be covered with coarse rip-rap to prevent erosion.

#### Placing of Class A Clay

After the river-cobble deposit has been breached with a clam-shell, the bedrock surface should be roughened sufficiently to insure a good bond of the corewall clay with the rock. The same goes for the eastern abutment of the dam, except that some actual excavation into the bedrock will be necessary to form an effective cut-off. Probably the bottom of the valley can be excavated in two sections in order to deal with the stream effectively. A modified wall-point system may be useful in dewatering the river-cobble deposit during excavation.

Special care should be taken to carry the Class A material westward into the spur to such a point that any loose pervious material such as found from 23½ to 26 feet in Hole 8 is cut off, and in addition either Class A or the best grade of B material should be used for the entire western part of the B section of the dam which wraps around the toe of the spur and extends well back of the projecting end of the spur. In this section of the upstream fill, excavation down to bedrock should be undertaken over a width of at least 30 feet. This excavation would not disturb the toe of the spur but would flank the spur on the north. Sufficient material should be stripped off the north side of the spur to get down to tight clay which will bond readily with the fill of the best grade of B material.

### Conclusions On The Explorations

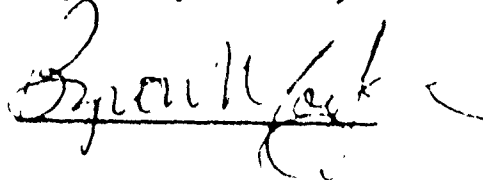
Exploration and borings made after submission of the preliminary geological report indicate decidedly unusual subsurface conditions in the area of the proposed structure. These conditions do not vitiate the site to any appreciable degree, but they rather indicate that some extra care must be taken to insure a safe and effective cut-off of circulation through the boulder deposit below creek level extending westward possibly onto the very tip end of the toe of the spur in which the west end of the dam will be founded.

The borrow material probably cannot be obtained without encountering boulders, but these should not be so numerous as to cause serious trouble in panning the Class A and B material. Some searching around in the extensive area of proposed borrow may be necessary to determine where a reasonably large quantity of good material can be obtained readily. The fact that the Class A material will expectably overlie the B clay will mean that strip panning rather than shovel excavation of the clay will be the more feasible.

In view of all the unexpected situations encountered, doubt may exist in some quarters as to the suitability of the site explored. It should be emphasized that no boring or other exploration work has revealed a condition that cannot be dealt with effectively and without any unusual expense. The worst condition, namely, the boulder deposit below creek level, is narrower at the proposed dam site than at any point farther upstream. Therefore, the amount of troublesome excavation will be less at the proposed site than at any other that might be utilized farther upstream. The alternative site farther upstream would entail penetration of river cobbles over a width of about 600 feet whereas at the proposed site the width is barely 220 feet.

It is believed that every possible adverse condition has been thoroughly investigated and the site found to be usable and the most desirable one in the valley of Stony Creek.

Respectfully submitted,



April 10, 1954

In Triplicate

APPENDIX V

REPORT ON EARTH RESISTIVITY SURVEYS



BYRON N. COOPER  
GEOLOGIST  
BOX 13  
BLACKSBURG, VIRGINIA

V. P. I. OFFICE  
8861 EXT. 380

RESIDENCE  
100 COUNTRY CLUB DRIVE  
TELEPHONE 8736

REPORT ON EARTH RESISTIVITY SURVEYS AT PROPOSED DAM SITE  
ON STONEY CREEK, NEAR BEDFORD, VIRGINIA

During three days in late November and early December, earth resistivity survey lines were run along lines 1-25 left, 0-50 left, and the Base Line for the purpose of determining the depth and profile of the contact between the soil and bedrock. Two additional depth determinations were made on the southwest side of the valley investigated. The locations of stations and profiles to bedrock are shown on the accompanying maps and sketches. The surveys were run along lines of the grid layout of the preliminary survey of the dam site, and some of the lines were extended beyond the limits of the preliminary survey, in order to prospect a larger area as a possible borrow area for Class A and Class B material.

The surveys suggest that in the area back of the old barn on the hillside the average depth to bedrock is about 12 feet, of which the upper 6 feet is probably good plastic compactible clay suitable for use in the core wall of the proposed dam. The lower 6 feet of the soil is probably too granular and rocky for use as Class A clay but would probably be satisfactory for Class B material. Considering the area prospected in this survey, approximately 18,000 cubic yards of Class A clay is believed to be recoverable from a tract of approximately 2.1 acres back of the barn on the hillside. This area could be enlarged westward to increase the yardage of clay to perhaps 25,000 yards, but probably not much more than this amount.

An additional source of borrow material is situated approximately 300 feet south-south east of the upstream end of the principal 2.1-acre borrow area, on the opposite side of the valley just uphill from an old orchard. Two depth determinations on thickness of the soil and mantle rock were made in this area, which indicate approximately 10 feet to bedrock.

The earth resistivity surveys do not, of course, determine the precise physical characteristics of the clays on top of the bedrock. Therefore it is suggested that three test pits be dug to determine the actual nature of the soil in critical parts of the area surveyed, as follows: (1) at 9-50 on line 0-50 left; (2) at 7-50 along line 1-50 left; and (3) at the edge of the orchard on the opposite side of the valley from the principal borrow area. These test pits will give a better understanding of the precise character of the clays in this general area and the proportion of Class A material to the other grades of material.

The tentative center line crosses the old lane at about 7-00 and from 450 to 700, the center line follows the crown of the projecting spur of ground that is to be used for anchoring the proposed dam. In this sector, i. e. downhill from the point where the centerline crosses the old lane, the earth resistivity survey line corroborates the previous inference I made that the spur is actually a hill of bedrock which makes an ideal cut-off for anchoring the proposed dam. The measurements made indicate that bedrock is within 6 to 14 feet of the surface. I believe that the results of the survey are sufficiently definite to take as established fact without the necessity of core-drilling, but one core hole somewhere along the centerline as finally determined should be put down to determine

the nature and occurrence of joints in the bedrock and the degree of disintegration of the bedrock in the core of the projecting spur on which the dam will be located.

The occurrence of bedrock at stream level along the present tentative center line seems sufficiently well established that it need not be core-drilled. However, at least two and possibly three horizontal core holes should be driven into the rock wall just to the east of the road. As previously mentioned, there are two rather well-developed systems of intersecting joints which conceivably could pipe impounded water around the east end of the dam if the sets were open joints. I believe it would be reasonable to restrict the amount of core drilling to 450 to 500 feet, in view of the considerable information already obtained.

It appears to me that the tentative center line should be rotated so that the section west of the creek is moved northward so as to lie entirely to the north of the old lane. When the final determination of the center line is made, it would be advisable to run an earth resistivity survey line right up the spur all the way to the highest projected water level. This could easily be done in one day's time.

As soon as the test pits are dug, I think I should examine the clay and collect samples for physical tests. It is possible that I am over-estimating the quantity of Class A material, but the curves plotted by the Moore's summation method and the square roots of the individual summation points indicated a break in the resistivity curve at a depth of 5 to 6 feet. This break is presumed to be the point of change from good clay down into granular residuum and weathered bedrock. If this presumption is not borne out by the sections exposed in the suggested

test pits, some revision will have to be made in the estimated quantities of Class A material available in the 2.1-acre tract that was prospected. The borrow areas are bound to contain a good many boulders of bedrock, but I do not believe that these will be sufficiently numerous to interfere with panning out the clay.

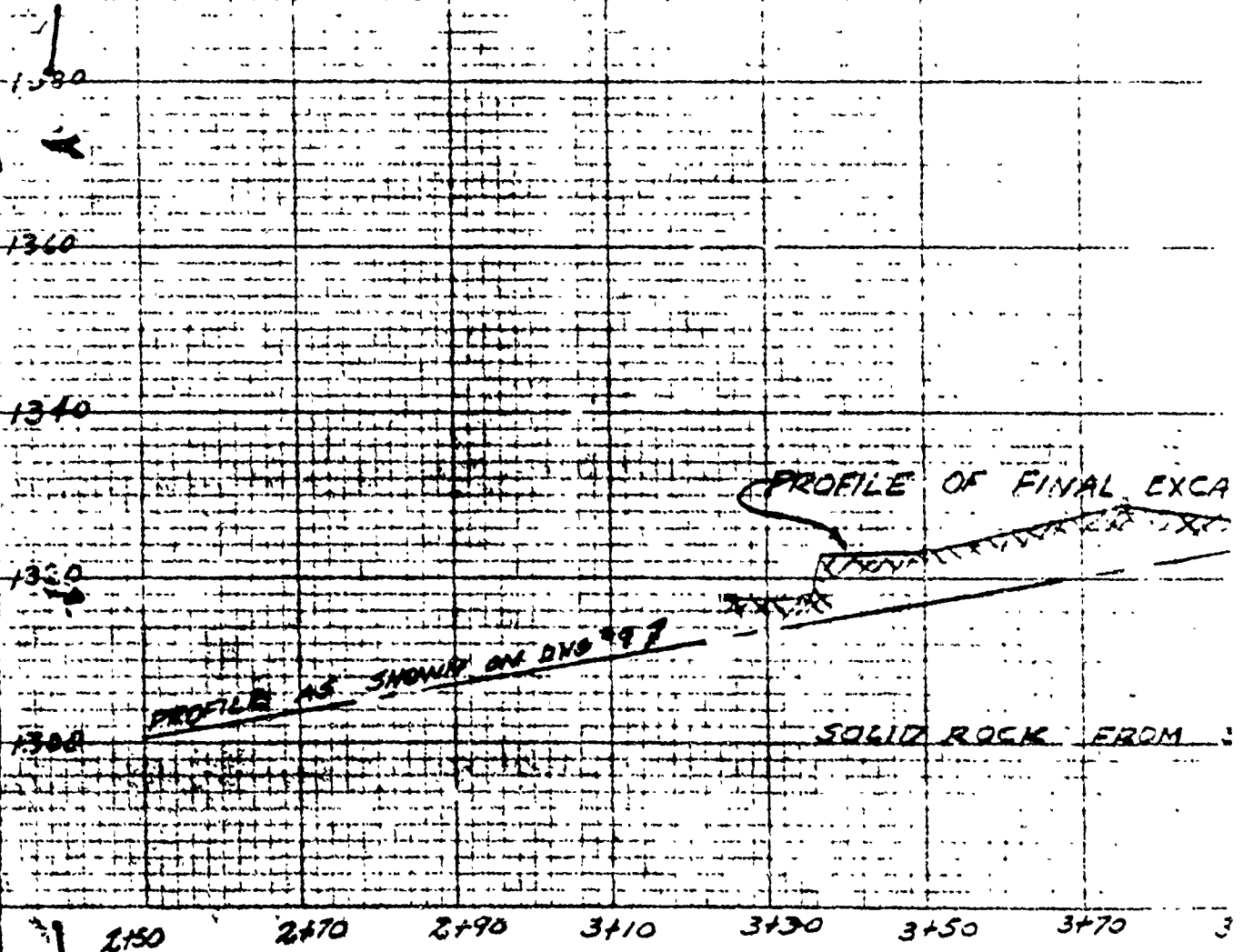
I did not anticipate that the work outlined to you would involve three days of work, but an additional day had to be spent in extending some of the lines a sufficient distance to outline that needed for borrow. The second day after Thanksgiving that Dr. Sears worked at the dam site, the helper to be supplied by the town failed to show up for work, and the surveying could not be done as rapidly as usual. I was a little disappointed at the relatively shallow depth of the clay down to the first break, i. e. the thickness of Class A material, and felt it advisable to go back to the area and map out extensions of the survey lines in order to explore a larger borrow area than we originally contemplated. Although I have not yet heard how much yardage will be required for the entire job, I am fairly certain that the main borrow area will not meet all your needs. Thus I felt it advisable to explore the area back of the old orchard on the opposite side of the valley.

The charges for the work done are enumerated on the accompanying sheet.

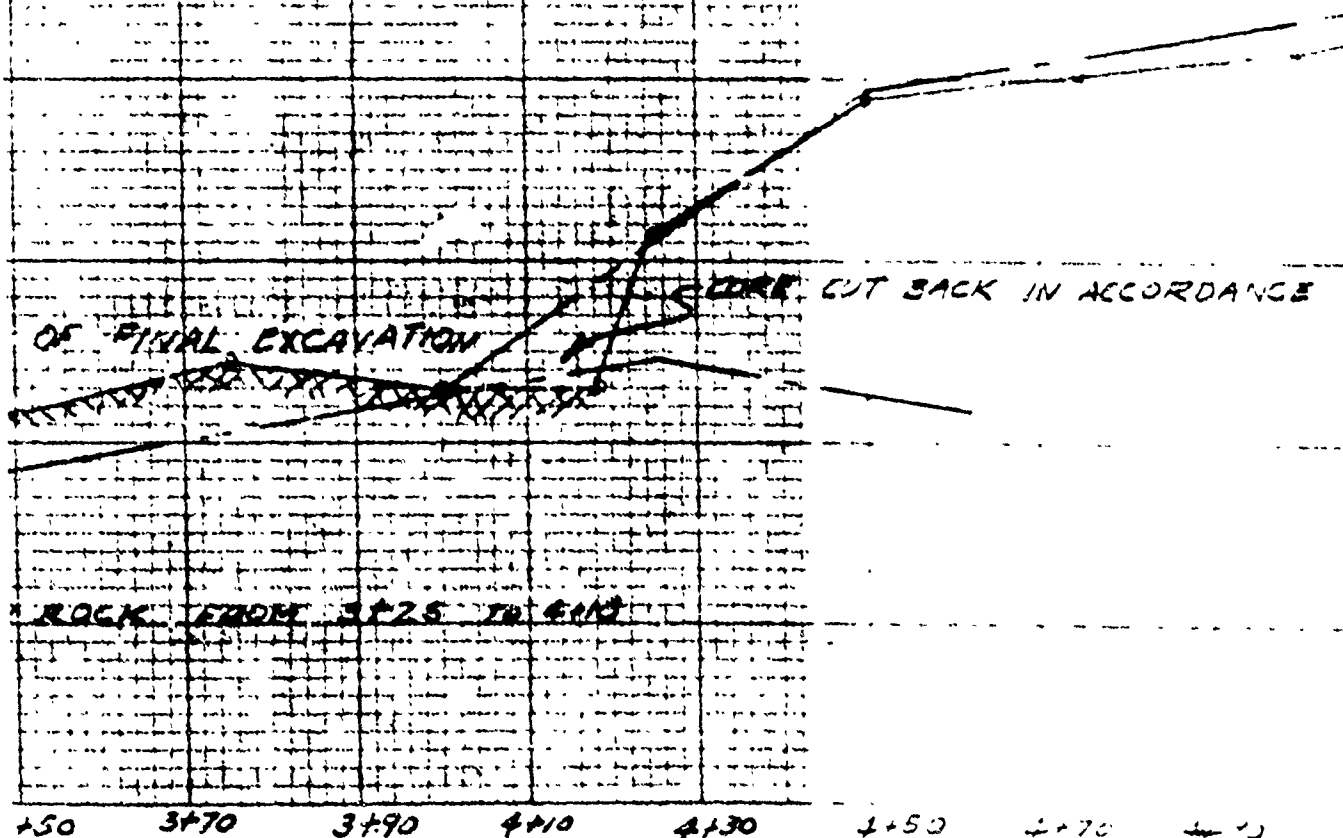
Respectfully submitted,

Byron Hooper

# SKETCH OF PROFILE



# OF PROFILE OF CLAY CORE



SCALE 1"=20'

# FALWELL WELL CORPORATION

COMPLETE WATER  
SYSTEMS

## WELL DRILLING

DIAMOND CORE  
CONTRACTORS

BOX 937      PHONE 2-7788  
3732 CAMPBELL AVENUE  
LYNCHBURG, VIRGINIA

April 8, 1954

Wiley and Wilson  
Church Street  
Lynchburg, Virginia

Attention: Mr. R. E. McNutt, Jr.    Re: Test Drilling - Proposed  
Dam Site - Bedford, Virginia

Dear Mr. McNutt:

On completion of our core drilling at the new proposed dam site for the town of Bedford we are below listing the formation encountered in our drilling.

### Test Hole No. 1

(We are referring to this as No. 1 as it was the first hole that we put down and is located half way up the hill between the creek and the top of the hill.)

From 5 ft. to 10 ft. - - Clay  
From 10 ft. to 42 ft. - - Sand and Boulders  
From 42 ft. to 45 ft. - - Top of Ledge Rock  
From 45 ft. to 50 ft. - - Ledge Rock

### Test Hole No. 2

(This hole was drilled on the edge of the creek itself and we found the following.)

From 5 ft. to 27 ft. - - Sand and Boulders  
27 ft. - - - - - Top of 3 ft. Bolder  
30 ft. - - - - - Bottom of 3 ft. Bolder  
31 ft. - - - - - Small Bolder and Sand  
32 ft. to 38 ft. - - - Ledge Rock

### Test Hole No. 3

(This hole was at the top of the hill and if you will recall this is the hole that we took the wash samples on which were not satisfactory and we moved off and we moved back and took the spoon samples which were satisfactory to Dr. Cooper and others.)

Cont'd.

"Central Virginia's Largest Drillers"

# FALWELL WELL CORPORATION

COMPLETE WATER  
SYSTEMS

## WELL DRILLING

BOX 937      PHONE 2-7786  
2722 CAMPBELL AVENUE  
LYNCHBURG, VIRGINIA

DIAMOND CORE  
CONTRACTORS

Wiley and Wilson

Page III

April 8, 1954

As you know all of the spoon samples that we took were turned over to Dr. Cooper and he took these samples back to Blacksburg. All of the rock core that we took on these jobs we now have in a core box in our ware house here and we will keep them in a safe place. Of course, upon your request we can deliver these cores to you at any time. We are holding in our files the logs from day to day which were turned in by the driller and we can go into greater detail if you so request.

We regret very much that this job took considerably longer than we had anticipated, however, as you know it was a rather difficult job and of course it is impossible to make as much time on a job of this kind as we would like.

Trusting that the above meets with your approval and any time that we can be of further service to you please do not hesitate to call on us.

Very truly yours,

FALWELL WELL CORPORATION

*W. Calvin Falwell*

W. Calvin Falwell

WCF/bt

50  
38  
65  
20  
25  
25  
38  
—  
261

"Central Virginia's Largest Drillers"



APPENDIX VI

STABILITY ANALYSES AND MINIMAL HYDRAULIC/HYDROLOGIC  
DESIGN CALCULATIONS

WATER WORKS IMPROVEMENTS  
BEDFORD VA.

Story Creek Reservoir

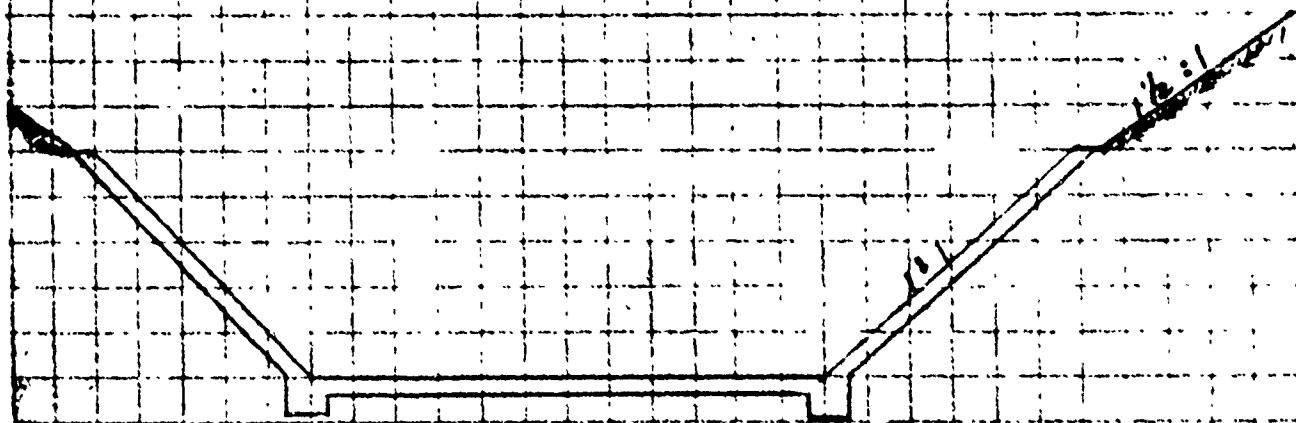
Drainage Area = 5.97 sq. mi.

Max Runoff = 2000 sec.-ft./sq. mi.

Max Flood Flow =  $2000 \times 5.97 = 11,940$  cu. ft./sec.

Use broad crested weir with Velocity Approach.

Not enough storage capacity in basin to take advantage of storage in reducing spillway capacity. Spillway must be designed to have discharge capacity equal to Max Flood Flow.



$$Q = 3.33 L h^{3/2}$$

Use  $h = 7'$

Use 60% Q for Paved Spillway, 40% Q for Emergency Spillway.

Paved Spillway

$$Q = .6 \times 11,940 = 7164 \text{ ft}^3/\text{sec}$$

$$= \frac{Q}{3.33 h^{3/2}} = \frac{7164}{3.33 \times 19.5} = \frac{7164}{64.7} = 110'$$

Q for Paved Spillway, 50%

$$n \cdot L = \frac{5970}{61.7} = 97'$$

Hydroelectric Investigation  
Bedford, Virginia

3/29/34  
R.C.D.

Design of Spillway for Stony Creek Dam

Procedure Used in design

1. Determined Max. Runoff (2000 cfs / sq. mi.)
2. Calculate Size of Entrance Weir.  
( $Q = 3.087 L (H + H_r)^{3/2}$ ) Reference - Hydroelectric Handbook - Creager & Justin: Page 125
3. Plot preliminary  $\pm$  to fit topography of the ground
4. Plot profile of exist. ground along profil  $\pm$  and determine approximate finish profile of spillway
5. Using approx. grades determine approx. width of channel using Bazin's Formula for flow in open channels. Reference - C.E. Handbook - Merriman & Higgins: Page 1344
6. Revise  $\pm$  if necessary to fit top. or to get proper width of earth betw. upstream face of dam and bottom of spillway cut. Repeat steps 3 & 4.
7. Check Width of Channel and depth of flow by Bernoulli's Theorem. & revise channel if necessary - Reference - Engineering for Dams - Creager, Justin & Hinds: Page 76 (See Also - Calculations for Mortimer's Dam)

Waterway Improvements  
Bedford, Virginia

Sheet 1 of 11  
P. 108  
D. 10

Design of Spillway for Stony Creek Reservoir

Estimated Maximum Flood Flow

Drainage Area 5.97 sq. mi.

Max. Run-off = 2200 cfs. / sq. mi.

Max. Flow =  $5.97 \times 2200 = 11,940$  cfs.

Design entrance to spillway as a broad crested weir.  
(Velocity at Approach = less than 4 ft/sec)

$$Q = C L H^{3/2} \quad (\text{Francis Formula})$$

$$\text{Use } C = 3.33$$

$$\text{Using } h = 6'$$

Design Permanent Spillway to take 1000 cfs. / sq. mi.

and Soft Plug to handle remaining 1000 cfs. / sq. mi.

$$Q = 5970 \text{ cfs.}$$

$$5970 = 3.33 L (6)^{3/2}$$

$$L = \frac{5970}{3.33 \times 18.7} = 121.96'$$

$$\text{Using } h = 8'$$

$$L = \frac{5970}{3.33 \times 22.63} = 79.22'$$

Design No. 1

$$L = \frac{5925}{3.83 \times 11.52} = 96.81$$

$$Q = 3.087 L (H - H_0)^{3/2}$$

$$H_0 = 0$$

$$L = \frac{Q}{3.087 (H)^{3/2}} = \frac{11.540}{3.087 \times 11.52} = 208.9$$

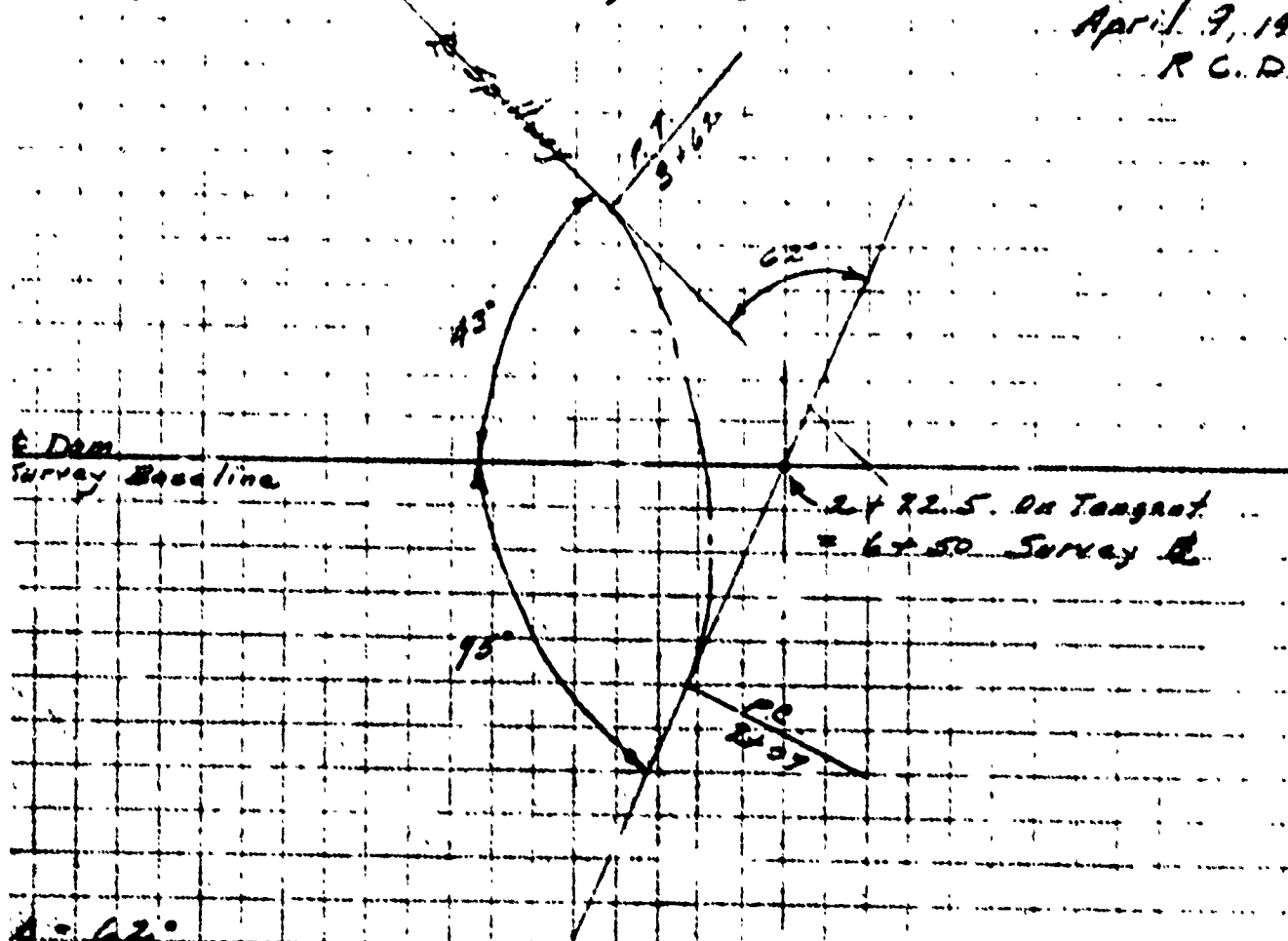
Reference: Hydraulics Handbook - Cramer & Justin  
Page 125

Use 200 wide Channel at 1/12

Depth: 7'

# Bedford, Va. - Spillway Alignment

April 9, 1954  
R.C.D.



$$\Delta = 62^\circ$$

$$D = 40'$$

$$R = \frac{5729.58}{40} = 143.24$$

$$T = R \tan \frac{\Delta}{2} = 143.24 \tan 31^\circ = 143.24 \times 1.60086 = 16.07'$$

$$L = \frac{100 \times 60}{40} = 155'$$

$$P.T. = 3162$$

$$2407$$

$$155$$

$$2193.07$$

$$P.C. = 2107$$

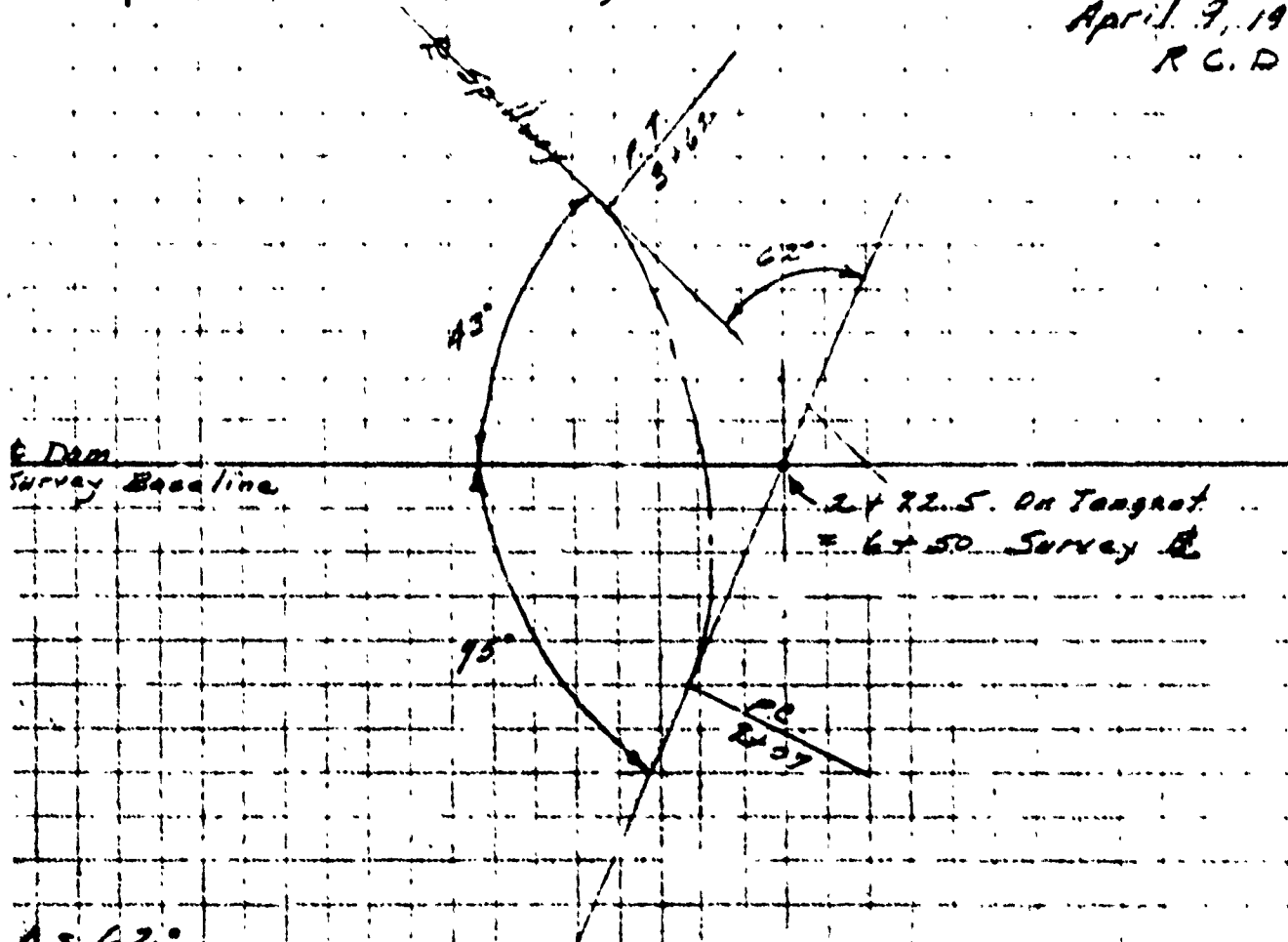
$$2193.07$$

$$155$$

$$2107$$

# Bedford, Va. - Spillway Alignment

April 9, 1954  
R.C.D.



$$\Delta = 62^\circ$$

$$D = 40'$$

$$R = \frac{5729.58}{40} = 143.24$$

$$T = R \tan \frac{\Delta}{2} = 143.24 \tan 31^\circ = 143.24 \times 1.60086 = 16.07'$$

$$L = \frac{100 \times 62}{40} = 155'$$

$$P.T. = 3+62.5$$

$$155'$$

$$P.C. = 2+07$$

$$2+07$$

$$155'$$

$$2193.07$$

$$155'$$

$$2193.07$$

Design - V<sub>2</sub> - Spillway

March 2nd 54  
R. G. D.

Flow - Prelim.

Reference - D. H. Chanson p. 1344

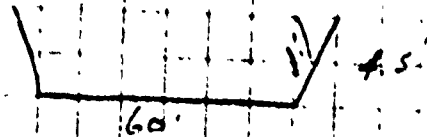
Bazin's Formula

Various  $C$  for  $m = 0.15$  for paved section  
 $m = 0.15$  unpaved

(1) Paved Section

$$Q = 11,940 \text{ cfs}$$

$$S = 175 \text{ ft}$$



$$\text{Assume } A = 60 \times 4.5 + (4.5)^2$$

$$= 270 + 20 = 290 \text{ sf}$$

$$WP = 60 \times 13 = 780 \text{ ft}$$

$$R = \frac{A}{WP} = \frac{290}{78} = 3.71 \quad C = 132$$

$$V = C \sqrt{RS} = 132 \sqrt{3.71 \times 175} = 132 \times 8.54 = 1127 \text{ fps}$$

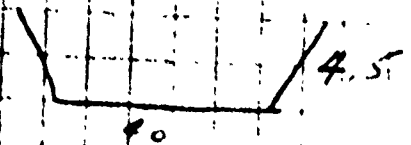
$$A = \frac{Q}{V} = \frac{11,940}{1127} = 10.6 \text{ sf}$$

N.G.

$$\text{Try } A = 40 \times 4.5 + (4.5)^2$$

$$= 180 + 20 = 200 \text{ sf}$$

$$WP = 40 \times 13 = 520 \text{ ft}$$



$$R = \frac{A}{WP} = \frac{200}{52} = 3.85 \quad C = 137.5$$

$$V = C \sqrt{RS} = 137.5 \sqrt{3.85 \times 175} = 137.5 \times 8.16 = 1121 \text{ fps}$$

$$A = \frac{Q}{V} = \frac{11,940}{1121} = 10.65 \text{ sf}$$

N.G.



$$V_{sc} = 0.88$$

$$A = 30 \times 4.5 + (4.5)^2 \cdot 13.5 \div 2 = 155 \text{ sf.}$$



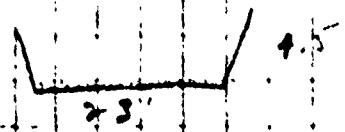
$$WP = 30 + 13.5 = 43.5$$

$$R = \frac{A}{WP} = \frac{155}{43.5} = 3.6 \quad C = 124.4$$

$$V = 124.4 \times \sqrt{3.6 \times 1.75} = 124.4 \times 2.47 = 308 \text{ f.p.}$$

$$A = \frac{Q}{V} = \frac{11700}{98.8} = 118.5 \text{ sf. N.G.}$$

$$A = 28 \times 4.5 + (4.5)^2 \cdot 10.4 \div 2 = 124 \text{ sf.}$$



$$WP = 28 + 13.5 = 41.5$$

$$R = \frac{A}{WP} = \frac{124}{41.5} = 3.0 \quad C = 123.8$$

$$V = C \sqrt{RS} = 123.8 \sqrt{3.0 \times 1.75} = 123.8 \times 2.29 = 283$$

$$A = \frac{Q}{V} = \frac{11500}{283} = 40.7 \text{ sf.}$$

Width of spillway at crest

$$\text{Broad Crested Weir} - Q = 3.087 L (H_1 + H_2)^{3/2}$$

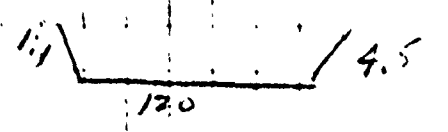
Reduce by 10-15%

Reference - Hyd. Davis P. 23

See Viscous Flow - Hydraulics - Handbook P. 140

Sta. 1+50

$$A = 120 \times 4.5 + (4.5)^2 \cdot 540 \div 2 = 560 \text{ sf.}$$



$$WP = 120 + 13.5 = 133.5$$

$$R = \frac{A}{WP} = \frac{560}{133.5} = 4.2 \quad C = 126 \quad (n = 0.015)$$

$$V = \frac{Q}{A} = \frac{11900}{560} = 21.3 \text{ f.p.}$$

$$V = C \sqrt{RS} \quad S = \frac{V^2}{C^2 R}$$

$$S = \frac{(21.3)^2}{(126.0)^2 \times 4.21} = \frac{453.69}{16,027.6 \times 4.21} = \frac{453.69}{67,476} = .0067$$

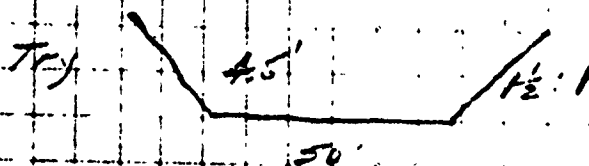
Check

$$Q = 3.367 \times 120 \times \left(4.5 + \frac{(21.3)^2}{2722.2}\right)^{3/2} = 370.44 \times (11.5)^{3/2}$$

$$Q = 14,497 \text{ cfs.} \quad \underline{\text{OK}}$$

Unpaved Channel

$$Q = 11,940 \text{ cfs}$$



$$A = 50 \times 4.5 + 4.5 \times 16 =$$

$$= 225 + 30 = 255 \text{ sf.}$$

$$WP = 50 + 16 = 66$$

$$R = \frac{A}{WP} = \frac{255}{66} = 3.86 \quad C = 88.3 \quad (m = 0.85)$$

$$S = .06$$

$$V = C \sqrt{RS} = 88.3 \sqrt{3.86 \times .06} = 88.3 \times 4.11 = 42.5$$

$$A = \frac{11,940}{42.5} = 280.9 \quad \underline{\text{N.G.}}$$

Try

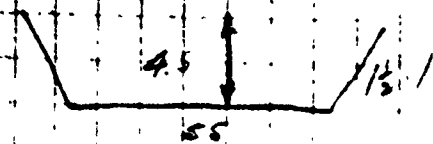
$$A = 55 \times 4.5 + 30 = 247.5 + 30 = 277.5$$

$$WP = 55 + 16 = 71$$

$$R = \frac{277.5}{71} = 3.91 \quad C = 88.6$$

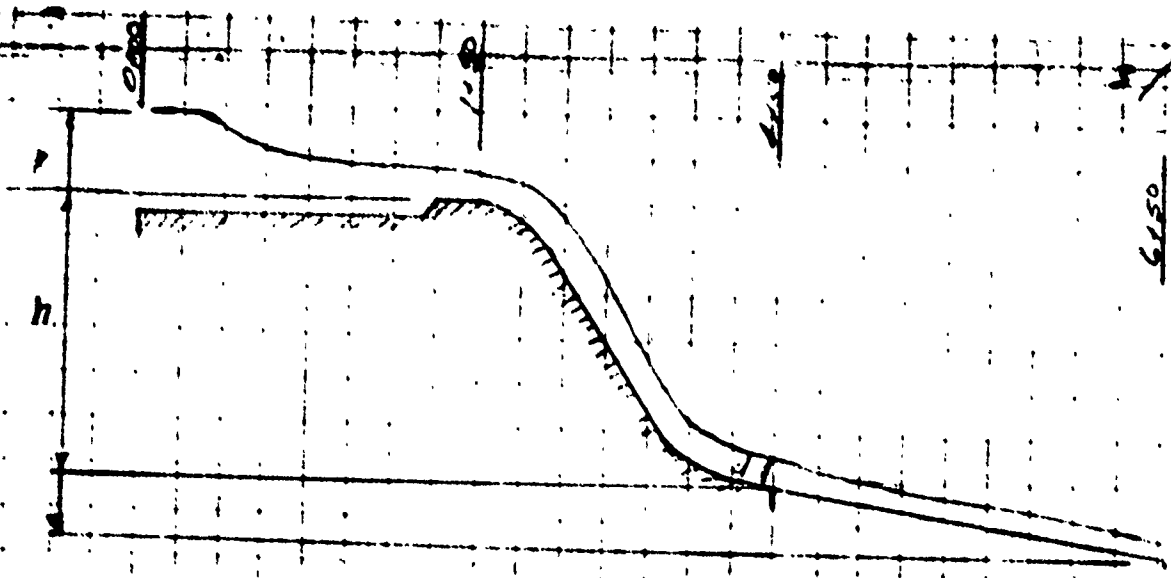
$$V = C \sqrt{RS} = 88.6 \sqrt{3.91 \times .06} = 42.9$$

$$A = \frac{11,940}{42.9} = 278.3$$



3/26/58

6150



$$H + h = d + \frac{2g^2}{29d^2} - \frac{12g^2h}{2.21d^2}$$

H = Depth of Crest = 7

h = height of dam

g = 32.2 / sec<sup>2</sup>

n = 0.015

g =  $\frac{Q}{w}$  cu ft / sec / ft channel

(1.) Find d of str 8150

H = 40.5

H = 7

w = 56

Q = 11,940

$$g = \frac{11,940}{56} = 213.2 \text{ ft/sec}$$

Try 

$$40.5 + 7 = d + \frac{2(213.2)^2}{2 \times 32.2 \times d^2} - \frac{(0.015)^2 (213.2)^2 (40.5)}{2.21 d^{1/2}}$$

$$47.5 = d + \frac{454.3}{32.2 d^2} - \frac{.000169 \times 45454 \times 40.5}{2.21 d^{1/2}}$$

$$47.5 = d + \frac{1411.6}{d^2} - \frac{3.47}{d^{1/2}}$$

$$47.5 d^2 - d^3 - 1411.6 = 3.47 d^{3/2}$$

2/25/57

$$47.5 d^{1.3} - d^{1.5} - 1411.6 d^{1.3} = 140.7 d^2$$

$$47.5 d^{1.3} - d^{1.5} - 1411.6 d^{1.3} = 140.7$$

Try  $d = 4$

$$47.5 (4)^{1.3} - (4)^{1.5} - 1411.6 (4)^{1.3} = 140.7$$

$$47.5 (101.59) - 406.38 - 1411.6 (3.35) = 140.7$$

$$4825.5 - 406.38 - 8,943.7 \neq 140.7$$

Try  $d = 5$

$$47.5 (5)^{1.3} - (5)^{1.5} - 1412 (5)^{1.3} = 140.7$$

$$" (21.3) - 1090 - " (1.6) = 140.7$$

$$12,118 - 1090 - 12,148 \neq 140.7$$

Try  $d = 6$

$$47.5 (6)^{1.3} - (6)^{1.5} - 1412 (6)^{1.3} = 140.7$$

$$" (390) - 2900 - 1412 (124) = "$$

$$18,525 - 2900 - 14,595 = 140.7$$

$$1440 \neq 140.7$$

$d < 6$

Increase size of channel to

$$d = \frac{11.11}{5} = 2.22 \text{ } 1.5 \text{ } 1.5$$

$$H-h = d + \frac{2.5^2}{25d^2} + \frac{0.49g^2h}{2.21d^{1/3}}$$

3/16/54

$$7-40.5 = d + \frac{2(120)^2}{25d^2} + \frac{(2.5^2)(120)^2(40.5)}{2.21d^{1/3}}$$

$$47.5 = d + \frac{59,601}{25d^2} + \frac{1,000,169 \times 39,601 \times 40.5}{2.21d^{1/3}}$$

$$47.5 = d + \frac{1230}{d^2} + \frac{122.6}{d^{1/3}}$$

$$47.5 d^{1/3} - d^{1/3} - 1230 d^{1/3} = 122.6 \quad (\text{See derivation above})$$

Try  $d = 5.0'$

$$47.5 \times 218 - 1070 - 1230 \times 18.6 = 122.6 \quad (\text{See above})$$

$$19,112 - 1070 - 10,578 = 122.6$$

$d > 5$

Try  $d = 5.5$

$$47.5 (5.5)^{1/3} - (5.5)^{1/3} - 1230 (5.5)^{1/3} = 122.6$$

$$(292) - 1600 - (9.7) = 122.6$$

$$13,870 - 1600 - 11,931 = 122.6$$

$$559 \neq 122.6$$

$d < 5.5$

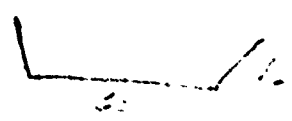
Sta. 4+50

Make channel 60' wide at base 5.5' deep

(2) Find  $d$  at Sta. 4+50

$$h = 52.5' \quad g = 199 \text{ cfs}$$

$$H-h = d + \frac{2.5^2}{25d^2} + \frac{0.49g^2h}{2.21d^{1/3}}$$



$$7+52.5 = d + \frac{1230}{d^2} + \frac{57 \times 10^4 \times 52.5}{2.21 d^{1/3}}$$

$$59.5 d^{1/2} - d^{1/2} - 1230 d^{1/2} = 159$$

3/26/54

Try  $d = 5'$

$$59.5 \times 2.15 - 1230 \times 2.15 = 159$$

(See above)

$$12,674 - 1230 - 10,578 = 159$$

$$1026 \approx 159$$

$$d < 5'$$

Try  $d = 4.5'$

$$59.5 (4.5)^{1/2} - (4.5)^{1/2} - 1230 (4.5)^{1/2} = 159$$

$$59.5 (150) - 670 - 1230 (7.4) = 159$$

$$8,925 - 670 - 9,114 \approx 159$$

$$d > 4.5$$

Sta 7+50

Note Chw. 1 30' wide at base, 5.0' deep

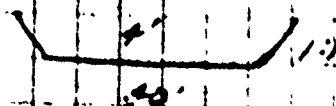
(3.)

Find  $d$  at Sta 4+50

$$h = 30'$$

$$q = \frac{4600}{40} = 292.5 \text{ cfs/ft}$$

Try



$$H + h = d + \frac{292^2}{2g d^3} + \frac{17^2 d^2}{2.21 d^{1/2}}$$

$$7 + 30 = d + \frac{2(292.5)^2}{2.5512 d^3} + \frac{(0.7)^2 (292.5)^2 \times 30}{2.21 d^{1/2}}$$

$$37 = d + \frac{89,102}{25.12 d^3} + \frac{600,69 \times 89,102 \times 30}{2.21 d^{1/2}}$$

$$37 = d + \frac{2,767}{d^3} + \frac{204.4}{d^{1/2}}$$

$$37 d^{1/2} - d^{1/2} - 2767 d^{3/2} = 204.4$$

(See Appendix - same)

$$37(4)^{1.5} - (4)^{1.5} - 2767(4)^{.5} = 204.4$$

$$37(10) - 400 - 2767(6.3) \setminus 204.4 \quad d > 4$$

try  $d = 5$

$$37(212) - 1070 - 2767(8.6) = 204.4 \quad (\text{see above})$$

$$7.8 - 1070 - 22,716 \neq 204.4 \quad d > 5$$

try  $d = 6$

$$37(390) - 2400 - 2767(10.4) \setminus 204.4 \quad d > 6$$

Increase Size of Channel to

$$q = \frac{11,940}{50} = 239 \text{ cfs/ft.}$$

$$37 = d + \frac{57.121}{32.2} + \frac{0.00168 \times 57.121 \times 30}{2.21 \text{ cfs}}$$

$$37 = d + \frac{1.774}{d} + \frac{2.31}{d^3}$$

$$37d^3 - d^3 - 1.774d^2 - 2.31 = 131 \quad (\text{see derivation above})$$

Try  $d = 6$

$$37(390) - 2400 - 177.4(10.4) = 131$$

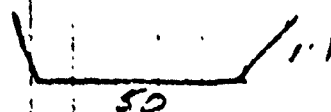
$$14,439 - 2400 - 18,450 \neq 131$$

Increase Size of Channel to

$$q = \frac{11,940}{65} = 184 \text{ cfs/ft.}$$

$$37 = d + \frac{12.37}{32.2} + \frac{0.00168 \times 12.37 \times 30}{2.21 \text{ cfs}}$$

$$37 = d + \frac{1.774}{d} + \frac{2.31}{d^3}$$



try d = 4

3/21

$$T(390) = 2400 - 1250(104) = 11$$

$$12480 = 2400 - 1250(104)$$

std. dev. 1000 = 1000

$$390 \times 1000$$

$$1250 \times 1000 = 1250000$$

(4)

Find d at 3.50

$$h = 12.5$$

$$2 = \frac{14900}{100} = 149 \text{ lbs/ft}$$

$$H + 12.5 = d + \frac{29^2}{55d^2} + \frac{11^2 92h}{2.21 d^2}$$

$$7 + 12.5 = d + \frac{(149)^2}{55.2 d^2} + \frac{.000169 \times (149)^2 \times 12.5}{2.21 d^2}$$

$$19.5 = d + \frac{2.30}{55.2 d^2} + \frac{.000169}{2.21 d^2}$$

$$19.5 = d + \frac{6.89}{d^2} + \frac{31.2}{d^2}$$

$$19.5 d^3 = d^3 + \frac{6.89}{d} + \frac{31.2}{d} = 21.2 \quad (\text{see derivation on p. 1})$$

Try d = 5

$$19.5(5)^3 = 1070 = 6.89(8.5) + 31.2$$

$$4.154 = 1070 - 39.25 = 21.2$$

d > 5

Try d = 5.5

$$19.5(5.5)^3 = 1070 = 6.89(11.7) + 31.2$$

$$5.514 = 1070 - 81.25 = 21.2$$

d = 5.5



Energy + Bed - y Channel

$$Q = \frac{11,940}{140} = 85.3 \text{ cfs / ft.}$$

$$19.5 \cdot d^{1.48} = \frac{85.3}{32.2} \Rightarrow \frac{19.5 \cdot d^{1.48} \cdot 32.2}{32.2} = \frac{85.3}{32.2}$$

$$19.5 \cdot d^{1.48} = \frac{85.3}{32.2}$$

$$19.5 \cdot d^{1.48} = \frac{85.3}{32.2} \Rightarrow \frac{19.5}{32.2} \cdot d^{1.48} = \frac{85.3}{32.2}$$

$$19.5 \cdot d^{1.48} = d^{1.48} - 440 \cdot d^{1.48} = 13.5$$

$$\text{Try } d = 5.5'$$

$$19.5(292) = 1600 - 440(9.7) = 13.5$$

$$5694 = 1600 - 4268 \approx 13.5$$

$$\text{Try } d = 6'$$

$$19.5(390) = 2400 - 440(10.4) = 13.5$$

$$7605 = 2400 - 4596 \approx 13.5$$

Sta. 2+50

Use Channel 140' wide & Deep

(5) Find d at Sta. 2+50

$$h = 1'$$

$$Q = \frac{11,940}{140} = 85.3 \text{ cfs / ft.}$$

$$7.1 = \frac{19.5 \cdot d^{1.48}}{32.2} \Rightarrow \frac{19.5 \cdot d^{1.48} \cdot 32.2}{32.2} = \frac{7.1 \cdot 32.2}{32.2}$$

$$8 = \frac{7.1 \cdot 32.2}{32.2} = \frac{228.14}{32.2}$$

$$8 = d = \frac{228.14}{32.2} = \frac{86}{32.2}$$

$$8(292) - 1600 - 226(d) = .56$$

try  $d = 5.5$

$$8(292) - 1600 - 226(5.5) = .56$$

$$2,336 - 1600 - 2,192 = .56$$

$d > 5.5$

try  $d = 6$

$$8(390) - 2400 - 226(10.4) = .56$$

$$3120 - 2400 - 2350 = .56$$

$d > 6$

Note: Due to the profile of this spillway this formula is not applicable at  $Sts \geq 50$ .

Using Broad-crested Weir Formula

$$Q = 3.087 L(H + H_c)^{3/2}$$

Assume  $H_c = 4.5$

$$A = 650$$

$$Q = 11,340 \text{ cfs}$$

$$V = \frac{Q}{A} = \frac{11,340}{650} = 17.4$$



$$L = 140'$$

$$\frac{V}{2.5} = \frac{17.4}{2.5} = 6.96$$

$$Q = 3.087 \times 140 (4.5 + 6.96)^{3/2}$$

$$= 3.087 \times 140 \times (9.76)^{3/2} = 13,376 \text{ cfs}$$

OK

Spillway (Bedford Water Works) 1/

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

$$V = \frac{4.49}{n} R^{2/3} S^{1/2}$$

$$V = \frac{1.486}{0.14} R^{2/3} S^{1/2}$$

$$V = 106.142 R^{2/3} S^{1/2}$$

$$V = 4.49 R^{2/3} S^{1/2} \text{ @ } 2:1 \text{ slope}$$

(ft/sec) V (max)	$R^{2/3}$	$S^{1/2}$	(sq ft) Wetted Area	(ft) Perimeter	(ft) Length	(ft) Height	Slope	ft/sec V (calc)	Note
54.6	3.56	0.21	954	142	127	7.5	.021	9.40	top of
171	2.293	0.45	200	58	50	4.0	.50	45	@ 2:1
194	2.591	0.45	250	60	50	5.0	.50	36	"
167	2.230	0.33	160	48	40	4.0	.50	52	"
224	2.79	0.45	450	87	75	6.0	.50	20 <sup>(1)</sup>	"
198	2.64	0.42	300	70	60	5.0	.50	30 <sup>(1)</sup>	"
49	3.16	0.21	1098	195	183	6.0	.021	22 <sup>(1)</sup>	top of

$$V = 106.142 \times 3.56 \times 0.21^{1/2}$$

$$V = 106.142 \times 3.56 \times 0.1449$$

$$V = 54.6 \text{ ft/sec}$$

to determine maximum flood flow for  
Stony Creek Sec. ft / sq. mi.

Name	Peak Flood flow cfs	date	drainage area	flood flow sq. ft / sq. mi.
Meadow Cr.				
@ Newcastle	700	1940	13.5	50.7
Johns Cr.				
@ Newcastle	5000	1935	105.0	75.5
Cataba Cr.				
@ Newcastle	3000	1949	39	72.2
Kerrs Cr.				
@ Lexington	23,000	1950	34	276.5 ←
Tye River				
@ Lexington	9670	1944	92	105.1
Other River				
@ Evinston	27,500	1937	325	84.6

$$Q = \frac{(2672 \times 1.5)}{1 + 230} + 151M \quad (\text{Design of Dams, Hammer Keady, p. 14})$$

$$Q = \frac{(36740 \times 1.5)}{597 + 230} + 15(597)$$

$$Q = (213)(597) = 1270 \text{ sec-ft}$$

$$Q = 1270 / 597 = 2.12 \text{ sec-ft / sq. mi.} \leftarrow$$

$$Q = \frac{10,000 \times 1.5}{1 + 230} \quad (\text{Water Supply Eng., Bechtel & Puley, p. 131})$$

$$Q = \frac{10,000 \times 1.5}{1 + 230}$$

$$Q = 2460 \text{ sec-ft / sq. mi.} \leftarrow$$

$$Q = 1500 \text{ sec-ft / sq. mi.}$$

Sperry Truck Drain

4.5 ft/sec  
1.5 ft

$$Q = 3.33 L (H)^{3/2}$$

$$12000 = 3.33 L H^{3/2}$$

$$L H^{3/2} = 3600$$

L	H <sup>3/2</sup>	H
295	14.7	6
378	9.54	4.5
171	20.5	7.5
133	27.0	9

$$Q = 3.33 L H^{3/2}$$

$$1500 \times 5.97 = 3.33 L H^{3/2}$$

$$L H^{3/2} = 2695$$

L	H <sup>3/2</sup>	H
283	9.54	4.5
183	14.70	6.0
127	20.5	7.5
100	27.0	9.0

Use 7.5' x 12' Spillway ←

Emergency Spillway will carry minimum  
of 500 sec-ft / sq. mi.

# BEDFORD, IN SPILL PAPER

From U.S. Geological map, it is approx 0.3" from dam to top of Snake Mountains

$$1" = 43,560' - \text{US. Geo map}$$

$$1" = 4,000'$$

from top of dam to top of Snake Mountains: { 600' top runway  
500' from bottom of dam to top of Snake Mountains: { 600' Terrace A  
650' Terrace B  
720' Terrace C  
800' d. conduit

C.B. #1

Q = AIR

$$L = 2.5$$

$$R_1 = 0.90 \quad R_2 = 0.80$$

$$A_1 = \frac{20 \times 600}{43,560} = \frac{12,000}{43,560} = 0.28 \text{ acres}$$

$$A_2 = \frac{20 \times 65}{43,560} = \frac{1,300}{43,560} = 0.03 \text{ acres}$$

$$Q = 0.28 \times 0.90 \times 2.50 + 0.03 \times 0.80 \times 2.50$$

$$= 0.63 \text{ cfs} \quad + .06$$

$$Q = 0.69 \text{ cfs}$$

C.B. #2

C.B. #3

C.B. #4

$$L = 2.5$$

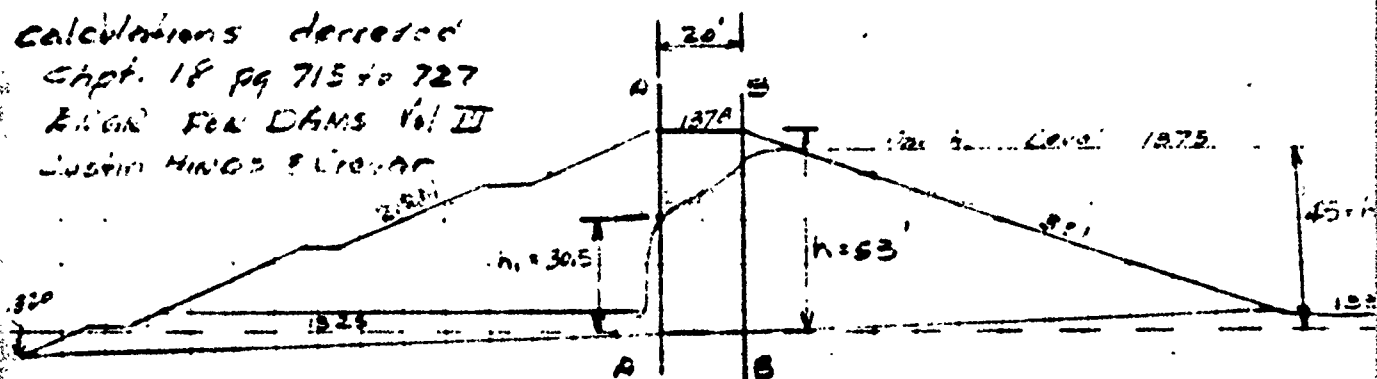
$$R = 80$$

$$A = \frac{20 \times 100}{43,560} = \frac{2,000}{43,560} = 0.05 \text{ acres}$$

$$Q = 0.05 \times 0.80 \times 2.5$$

$$= 0.10 \text{ cfs}$$

Calculations corrected  
 Chpt. 18 pg 715 to 727  
 EIGER FOR DAMS Vol II  
 JUSTIN HINDS & Co. Inc.



Take HORIZONTAL PLANE AT 1325 (Mid. Pt. of 1330 & 1320)

Assume  $\phi = 24^\circ$   
 $\tan \phi = .445$

Void Ratio = 0.40  
 $n = \frac{0.40}{1.40} = 0.29$

Wt of Material in DAM

Assume DRY Weight =  $62.5 \times 2.65 \times (1 - 0.29) = 118 \text{ ft}^3$   
 Saturated " =  $118 \text{ ft}^3 + (62.5 \times 0.29) = 136 \text{ ft}^3$   
 Submerged " =  $136 \text{ ft}^3 - 62.5 = 73.5 \text{ ft}^3$   
 Moist " Assume =  $122 \text{ ft}^3$

From calculation & plotting of seepage line

65% of x-sect. submerged =  $0.65 \times 136 = 88.4 \text{ ft}^3$   
 35% of x-sect. moist =  $0.35 \times 122 = 42.7 \text{ ft}^3$   
 Average unit effective wt of x-sect. =  $90.4 \text{ ft}^3$

AREA of 1' wide STRIP

$$\frac{20 + 1336}{2} \times 53 = 9450 \text{ ft}^2$$

EFFECTIVE WT of 1' STRIP

$$9450 \text{ ft}^2 \times 90.4 \text{ ft}^3 = 854,880 \text{ ft}^3 \text{ OR } 427 \text{ TONS}$$

$\tan \phi = .445$  without cohesion

RESISTING FORCE AGAINST HEADWATER PRESSURE

$$R = 427 \times 0.445 = 190 \text{ TONS}$$

$$\text{HEADWATER PRESSURE} = \frac{wh^2}{2} = 62.5 \times \frac{30.5^2}{2}$$

$$= 28,125 \text{ ft}^3 \text{ OR } 44.0 \text{ TONS}$$

$$R_d = W_{ed} \times \tan \phi + c b_i$$

FORMULA 7 pg 713

$$= 466,000 \times .445 + 0$$

$$= 207,000 \text{ or } 103.5 \text{ TONS} \leftarrow$$

$$F_d = \frac{103.5 \text{ TONS}}{42.95 \text{ TONS}} = 2.42 \text{ ok}$$

AVERAGE UNIT SHEAR

$$S_d = \frac{H_d}{b_i} \quad \text{FORMULA 2 pg 713}$$

$$= \frac{42.3 \text{ TONS}}{156.5} = 0.27$$

Max shear

$$S_{md} = 2 S_d \quad \text{FORMULA 3 pg 713}$$

$$= 2 \times 0.27$$

$$= 0.54 \text{ TONS}$$

Max shear occurs at 0.4 bd d. stream  
h @ 0.4 bd = 31.5

Unit shear strength at this pt

$$= 31.5 \times 1.445 = 45.4 \text{ or } 50.5 \text{ TONS} / 1.775$$

$$F_s = \frac{50.5}{54} = 1.49 \text{ ok}$$

Down stream Portion of DAM OK  $\leftarrow$  slope 1:2.5

HORIZONTAL SHEAR IN UPSTREAM PORTION  
Sudden Drawdown Worst Condition.

$$H_u = \frac{h^2 w_s \tan^2(45 - \frac{1}{2}\phi)}{2} \quad \text{Formula 4 pg 719}$$

$$w_s = 136 / 41^2$$

$$h = 53$$

$$H_u = \frac{(53)^2 (136) \tan^2 33^\circ}{2} = \frac{352,000 \times .422}{2}$$



$$F = \frac{190}{44} = 4.3 \text{ or}$$

Average pressure along 1' strip

$$p = \frac{44.0}{356} = 1.27 \text{ tons/ft}^2$$

$$F_s = \frac{16.2}{44} = 0.37$$

OK DAM IS OK FOR HEADWATER PRESS.

Ans. shear due to headwater pressure

$$S = \frac{44.0}{356} = 0.13 \text{ tons/ft}^2$$

HORIZONTAL SHEAR IN DOWNSTREAM PORTION  
LINE A-A

FORMULA - 1 pg. 710

$$H_d = \frac{h^2 w \tan^2(45 - \frac{1}{2}\phi)}{2} + \frac{w_w h_1^2}{2}$$

$$w_w = 62.5$$

$$h = 53$$

$$h_1 = 31.5 \text{ (ht. of scarp line at LINE A-A)}$$

$$w = \frac{31.5 \times 73.5}{53} + \frac{21.5 \times 122}{53} = 43.6 + 49.4 = 93.0 \text{ lb/ft}^2$$

$$H_d = \left[ \frac{(53)^2 (93.0) \tan^2(45^\circ - 12^\circ)}{2} \right] + \frac{62.5 (31.5)^2}{2}$$

$$= \frac{261,000 \times \tan^2 33^\circ}{2} + 30,900 = 130,500 \times 0.422 + 30,900$$

$$H_d = 85,900 \text{ or } 42.95 \text{ Tons} \leftarrow$$

RESISTING FORCE

$$\text{AREA} = \frac{156.5 \times 53}{2} = 4150 \text{ ft}^2$$

$$\text{UNDER SCARP LINE} \quad \text{APPROX } \frac{1}{2} = \frac{4150}{2} = 2075$$

$$\text{SUBMERGED} = 2075 \times 73.5 = 152,500$$

$$\text{MOIST} = \frac{2075 \times 122}{4150} = 61,500$$

$$4150$$

$$214,000 = W_d$$

$$H_u = 90,600 \text{ }^{\circ} \text{ or } 40.3 \text{ TONS}$$

RESISTING FORCE

$$\text{AREA} = \frac{159 \times 53}{2} = 4200 \text{ }^{\circ} \text{ }^2$$

$$W_{eu} = 4200 \text{ }^{\circ} \text{ }^2 \times 73.5 = 309,200 \text{ }^{\circ} \text{ or } 154.0 \text{ TONS}$$

$$R_u = 154 \times 0.445 = 68.6 \text{ TONS}$$

$$F_u = \frac{68.6}{40.3} = 1.7 \text{ should be } 2.0 - \text{ so too low}$$

$$S_u = \frac{40.3}{159} = 0.26$$

$$S_{mu} = 2 \times 0.26 = 0.52 \text{ (at pt 0.46u upstream)}$$

Unit shear strength at this pt.  
h = 31.5

$$S_u = 31.5 \times 1 \times 73.5 \times 0.445 = 1030 \text{ }^{\circ} \text{ or } 0.52 \text{ TONS}$$

$$F_u = \frac{0.52}{0.52} = 1.0 \text{ too low}$$

UPSTREAM PORTION SHEAR VALUES TOO HIGH  
TRY 1:3 1/2 SLOPE UPSTREAM

$$L_u = 35 \times 53 = 185.5$$

$$h = 53$$

$$H_u = 40.3 \text{ TONS}$$

RESISTING FORCE

$$\text{AREA} = \frac{185.5 \times 53}{2} = 4910 \text{ }^{\circ} \text{ }^2$$

$$W_{eu} = 4910 \text{ }^{\circ} \text{ }^2 \times 73.5 = 360,000 \text{ }^{\circ} \text{ or } 180.0 \text{ TONS}$$

$$E_u = 180.0 + 445 + 0$$

$$= 80.0 \text{ TONS}$$

$$F_u = \frac{20.5}{40.3} = 1.95 \text{ ok as rip rap screen above seepage was not figured in calculations}$$

$$S_u = \frac{40.3}{185.5} = 0.22 \text{ TONS}$$

$$S_{um} = 210.22 = 0.44 \text{ TONS}$$

$$S_s = 31.5 \times 1 \times 73.5 \times 0.445$$

$$= 0.52 \text{ TONS}$$

$$F_s = \frac{.52}{.44} = 1.18 \text{ ok above unity}$$

REG'D

UPSTREAM SLOPE -  $1:3\frac{1}{2}$

DOWNSTREAM SLOPE -  $1:2\frac{1}{2}$

# BEDFORD, VA.

29 5'-0" x 6'-0" Box Conduit for Conduit from  
Lake Tower to run under dam. - Design for 100 cfs.

At Conduit Entrance  
Figure on 10' head.

$$H = \frac{Q^2}{29a^2} (1+k) \rightarrow \text{Pg 101 Art 5}$$

Hydro-Electric Handbook  
Gregor & Justini.

$$Q^2 = \frac{29a^2 H}{1+k}$$

$$Q = \sqrt{\frac{29a^2 H}{1+k}} = \sqrt{\frac{2 \times 32.2 (5 \times 6)^2 10}{1 + 0.56}} = \sqrt{\frac{64.4 (900) 10}{1.56}}$$

$$= \sqrt{\frac{579,600}{1.56}}$$

$$= \sqrt{372,000}$$

$$Q = 610 \text{ cfs or } 102 \text{ cfs/mi.}$$

Within Conduit.

$$\text{Slope} = 3.5\% \quad 35' / 1000'$$

$$Q = AV$$

$$V = \frac{1.486}{n} R^{4/3} S^{1/2}$$

$$R = \frac{A}{wp} = \frac{30}{22} = 1.36$$

$$S = .035$$

$$Q = [1.5] \left( \frac{1.486}{.075} \right) (1.36)^{4/3} (.035)^{1/2}$$

$$= 30 \times 114 \times 1.06 \times 0.19$$

$$= 30 \times 23$$

$$= 690 \text{ cfs or } 115 \text{ cfs/mi.}$$

flowing full

3-16" inlets  
1-24" inlet

$$Q_T = Q_1 + Q_2 + Q_3 + Q_4$$

$$= 0.7a_1 \sqrt{2gh_1} + 0.7a_2 \sqrt{2gh_2} + 0.7a_3 \sqrt{2gh_3} + 0.7a_4 \sqrt{2gh_4}$$

$$= 0.7(1.39)(8.1) \sqrt{h_1} + 0.7(1.39)(8.1) \sqrt{h_2} + 0.7(1.39)(8.1) \sqrt{h_3} + 0.7(4) \sqrt{h_4}$$

$$= 7.9 \sqrt{h_1} + 7.9 \sqrt{h_2} + 7.9 \sqrt{h_3} + 22.7 \sqrt{h_4}$$

$$= 7.9 [\sqrt{h_1} + \sqrt{h_2} + \sqrt{h_3}] + 22.7 \sqrt{h_4}$$

ELV WATER LEVEL	$h_1$ 16"	$h_2$ 16"	$h_3$ 16"	$h_4$ 24"	$Q_1$ 16"	$Q_2$ 16"	$Q_3$ 16"	$Q_4$ 24"	$Q_T$
1368 6 6 6 2	8	18.5	29	36	22.4	34	42.5	136.2	235.4
1360 5 6 8 2	0	10.5	21	28	0	25.6	36.2	120	182.4
1350 5 6 4 2	0	0.5	11	18	0	5.6	26.2	96	128
1340 8 6 4 2	0	0	1	8	0	0	7.9	64	72
1330 2	0	0	0	2	0	0	0	32	32
1320	0	0	0	0	0	0	0	0	0

Use 30" square sluice gate

$$Q_4 = 30.69$$

$$Q = 0.7(6.25)(8.1) \sqrt{h_4}$$

$$= 35.4 \sqrt{h_4}$$

$$Q_4 = 30.69$$

$$Q_1 + Q_2 + Q_3$$

$$3-16" Pops$$

$$99$$

$$213$$

$$31$$

$$167$$

$$24$$

$$147$$

$$2$$

$$32$$

$$150$$

$$1$$

$$118$$

$$130$$

$$11$$

$$77$$

$$3$$

$$30$$

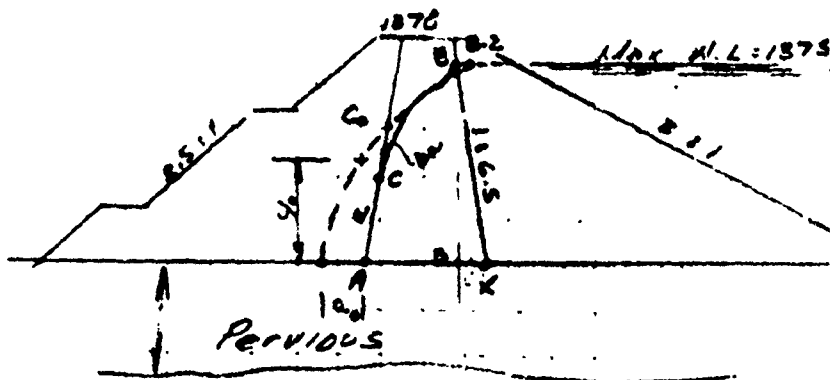
$$0$$

$$40$$

Cross Section Revised  
Seepage Line Recalculated

ENGR FOR DAMS Vol. II  
Cragg & Justin  
Pg. 665 on

$$\begin{aligned} BX &= 8' = m \\ B \cdot B_2 &= 0.3m = 2.4' \\ AB_2 &= 26.5' = d \end{aligned}$$



$$X = \frac{y^2 - y_0^2}{240}$$

$$\begin{aligned} y_0 &= \sqrt{h^2 + d^2} - d \\ &= \sqrt{(50)^2 + (26.5)^2} - 26.5 \\ &= \sqrt{2500 + 703} - 26.5 \\ &= \sqrt{3203} - 26.5 = 56.6 - 26.5 \\ &= 30.1 \end{aligned}$$

$$X = \frac{y^2 - y_0^2}{240}$$

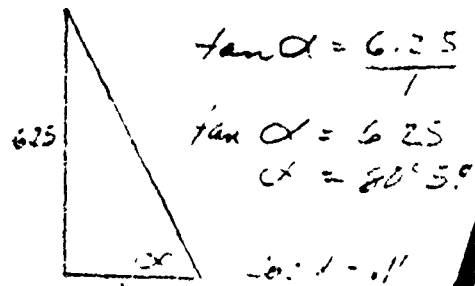
$$\text{then } y^2 = 240X + y_0^2$$

$$\begin{aligned} y &= \sqrt{240X + y_0^2} \\ &= \sqrt{2(1)(30.1) + (30.1)^2} \\ &= \sqrt{60.2X + 905} \end{aligned}$$

X	y	y
0	905	30.1
5	1206	34.7
10	1507	38.8
15	1808	42.5
20	2109	45.9
25	2410	49.2
26.5	2500	50

$$a_0 = \frac{y_0}{2} = \frac{30.1}{2} = 15.05'$$

$$\begin{aligned} a + \Delta a &= \frac{y_0}{1 - \cos \alpha} = \frac{30.1}{1 - 1.5787} \\ &= \frac{30.1}{.84213} = 35.7 \end{aligned}$$



$$\Delta a = c(a + \Delta a)$$

$$c = 0.275$$

$$\Delta a = 0.275 \times 35.7$$

$$\Delta a = 9.8$$

$$a + \Delta a = 35.7$$

$$a = 35.7 - 9.8$$

$$a = 25.9$$

Seepage limit at downstream toe  
5' sand water.

$$q = k \sqrt{d^2 + h^2} - d$$

$$k = 0.4 \times 10^{-4}$$

$$= .00004130.1$$

$$= .001204$$

$$= .0012 \text{ ft}^3/\text{min}$$

$$L = 25.6'$$

$$q = \frac{k}{2} \left( \frac{h^2 - e^2}{L} \right) = \frac{.00004}{2} \left( \frac{50^2 - 25.5^2}{L} \right)$$

$$q = \frac{.00002(2500 - 650)}{L}$$

$$L = \frac{.00002(1850)}{.0012} = \frac{.03700}{.0012}$$

$$L = 31'$$

$$q = k' \left( \frac{h'}{L'} \right) \left( 5 + \frac{h'}{2} \right)$$

$$q = 0.025 \frac{h'}{31} \left[ 5 + \frac{h'}{2} \right]$$

$$q = .0008 h' \left[ 5 + \frac{h'}{2} \right]$$

$$q = .004 h' + .0004 h'^2$$

$$.0004h'^2 + .004h' = .0012$$

$$h'^2 + 10h' = 3$$

$$h^2 + 10h + 25 = 28$$

$$(h+5)^2 = 28$$

$$h+5 = \sqrt{28}$$

$$h = 5.3 - 5$$

$$h = 0.3$$

1319

5 T.W.  
1324

.3  
1324.3



## APPENDIX VII

### REFERENCES

1. HEC-1DB Flood Hydrograph Package, (Hydrologic Engineering Center, U. S. Army Corps of Engineers, July 1978).
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4. "Recommended Guidelines for Safety Inspection of Dams," Department of the Army, Office of the Chief of Engineers, Washington, D.C.